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Power Division

PROCEEDINGS OF THE
AMERICAN SOCIETY OF CIVIL ENGINEERS

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Journal of the

POWER DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

April, 1959

Papers

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ENGINEERING ASPECTS OF THE SANTA SUSANA NUCLEAR POWER STATION

Dallas I. Downs,[1] George E. Deegan[2] and Robert F. Boggus,[3] J. M. ASCE

ABSTRACT

Description with charts and pictures of a sodium-cooled reactor with associated power plant near Los Angeles, including construction details; reactor design; radiation shielding; fuel and waste handling; water treatment; and coolant handling problems.

Method of plant start-up and operating experience to date are included.

1. INTRODUCTION

In mid-1954, North American Aviation, Inc., entered into a program with the U. S. Atomic Energy Commission aimed at expanding the known area of information on the sodium graphite reactor concept. One phase of this program consisted of the design, construction, and operation of a 20 MW (thermal) Sodium Reactor Experiment (SRE). An analysis of construction methods and materials, operating procedures, fuel element performance, and maintenance costs for the SRE is expected to provide much of this required information.

2. Site Location

A. General Considerations

The considerations involved in locating such a plant were necessarily more complex than those required for a normal manufacturing or power

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facility. Prime importance had to be placed upon achieving public acceptance of a nuclear reactor program, as well as upon conforming to all existing codes and regulations. North American Aviation began a search for a suitable site which would provide the answer to their threefold problem of:

1. Isolating the facility from heavily populated areas.
2. Providing sufficient water, power, and other utilities for an extensive construction program and an extended research effort.
3. Maintaining convenient engineering contact with the corporate offices in the Los Angeles area.

After careful review it was decided to locate the plant within an area adjacent to the existing North American Aviation field laboratory in the Simi Hills. The site is approximately 36 miles northwest of downtown Los Angeles.

B. Description of Santa Susana Area

The Simi Hills are primarily composed of marine sediments consisting of tightly cemented sandstones and shales of the Upper Cretaceous Age. These layers are relatively impervious and are tilted with a resulting strike of 325^{0} and a dip of approximately 22^{0}. The exposed strata have a maximum thickness of about 30 feet. In many places they are fractured and large monoliths have broken loose and are scattered on the hillsides. The shales tend to decompose rather rapidly when exposed to the weather.

Since the site is located approximately 16 miles from the San Gabriel fault the area can be considered as free from seismic disturbances as any in the vicinity of Los Angeles. There are several inactive faults in the area, however, and at depths of 500 - 1000 feet the fissures and fractures of these faults contain a water bearing stratum. Wells drilled into this pervious material deliver an average of 155 g.p.m.

The hills surrounding the reactor (Fig. No. 1) are too rough for any useful purpose and it is this barren area which provides the required isolation for the reactor. The site is unique in that it has the necessary security requirements and yet is located within convenient commuting distance of a major metropolitan community and its industrial and scientific resources.

A two-lane asphalt surfaced road connects the field laboratory with the community of Canoga Park. Improvement of two miles of dirt road provided access to the reactor site. Electrical power from Southern California Edison Company and water from wells in the immediate area fulfilled the site requirements.

3. Design of Reactor

A. General Arrangement

In the final design of the SRE the top of the reactor, and the operating floor of the reactor building are at grade level. The reactor core and all of the associated equipment and piping are below grade. The control room, cranes and handling equipment are above grade.

B. Construction

Site clearing and foundation excavation began early in 1955. (Fig. No. 2) Preliminary borings showed the entire site to be underlaid with the sandstone

FIGURE NO. 1

FIGURE NO. 2

of the area. Foundations for all major components such as the reactor, the hot cells, the bridge cranes, and the primary vaults rest on this rock.

C. Shielding and Containment

Reinforced concrete was extensively used for the reactor building substructures. Dense concrete, Sp. Gr. 3.6, with graded magnetite iron ore as aggregate, was utilized as radiation shielding around the core, in the blocks covering the pipe galleries, and in the walls inclosing the hot cells. The cover over the reactor core provides the required shielding as well as access for fuel and control elements. A cylindrical steel shell, stepped to prevent radiation streaming, was filled with magnetite coarse aggregate (min. size 3/8") and then intrusion grouted using magnetite fines, and cement. Additional shielding materials employed at the SRE include, steel plate, lead shot, and lead glass windows.

With low operating pressures, and coolant-fuel compatibility, it was not necessary to provide a pressure containment vessel for the reactor. There are, however, three separate positive inclosures of the radioactive sodium. (Fig. No. 3) The cavity formed by the concrete biological shield is lined with a carbon steel tank. An outer tank of alloy steel fits within this cavity liner. A stainless steel tank contains the core and is supported within the outer tank. A steel thermal shield fills the annular space between the core tank and the outer tank. Thermal insulation fills the space between the outer tank and the cavity liner.

D. Superstructure

Spread concrete footings support the structural steel frame of the reactor building. Separate steel columns supported on combined footings carry the 75 ton bridge crane. Concrete panels were precast on the site and tilted into place to form the sides of the reactor structure. Caulking between the panels, and seals around openings provide a secondary inclosure in the event of an accidental radioactive release within the building. The inclosure philosophy was continued in the design of the ventilating system. A slight negative pressure is maintained in the reactor building and discharged air is first passed through absolute filters to remove particulate matter.

E. Fuel Handling and Storage Facilities

Fuel handling facilities are required in order to remove or replace any of the SRE core elements. The fuel cask must provide for radiation shielding and gas containment during the handling operation. (Fig. No. 4)

The SRE fuel cask is a cylindrical container, 35 feet in overall height and weighing 50 tons. It has a maximum thickness of 9 inches of lead shielding near its base. Cooling is provided by means of a blower which circulates air between the cask body and the lead shield.

The fuel cask is supported by a carriage which rides on rails on the bridge of the 75 ton crane. By this means it can be positioned at any point within the reactor building.

In operation, the cask is located over a selected fuel plug. A seal mechanism is lowered to effect a gas-tight seal with the reactor vessel. A grapple assembly is lowered which engages the fuel element plug. The plug with attached fuel element is then raised into the cask. The interior mechanism is

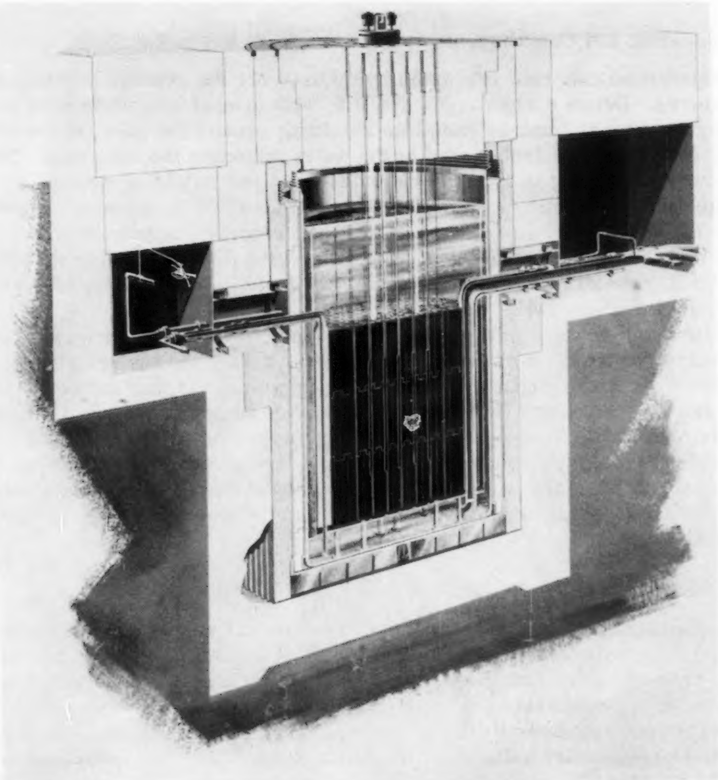


FIGURE NO. 3

then rotated and an element is lowered, to take the place of the element removed. The lower section of the cask is then sealed and the gas seal mechanism raised.

The removed element is then transported to the wash cells. The element is washed with demineralized water and dried by evacuation of the cell.

Ninety-six fuel storage cells are provided. These cells are set in heavy concrete and provided with cooling coils to remove the "afterglow" heat.

F. Waste Disposal

As a result of fuel washing and possible activation of gas in the galleries, it is necessary to provide systems to handle radioactive by-products.

Radioactive liquid is pumped into holdup tanks which are contained in shielded vaults external to the reactor building. (Fig. No. 5) The liquid is held in these tanks to permit radioactive decay. It is then drained into one of two 5000 gallon storage tanks. These tanks are emptied periodically, the liquid is placed in drums and incased in concrete. The containers are disposed of at sea.

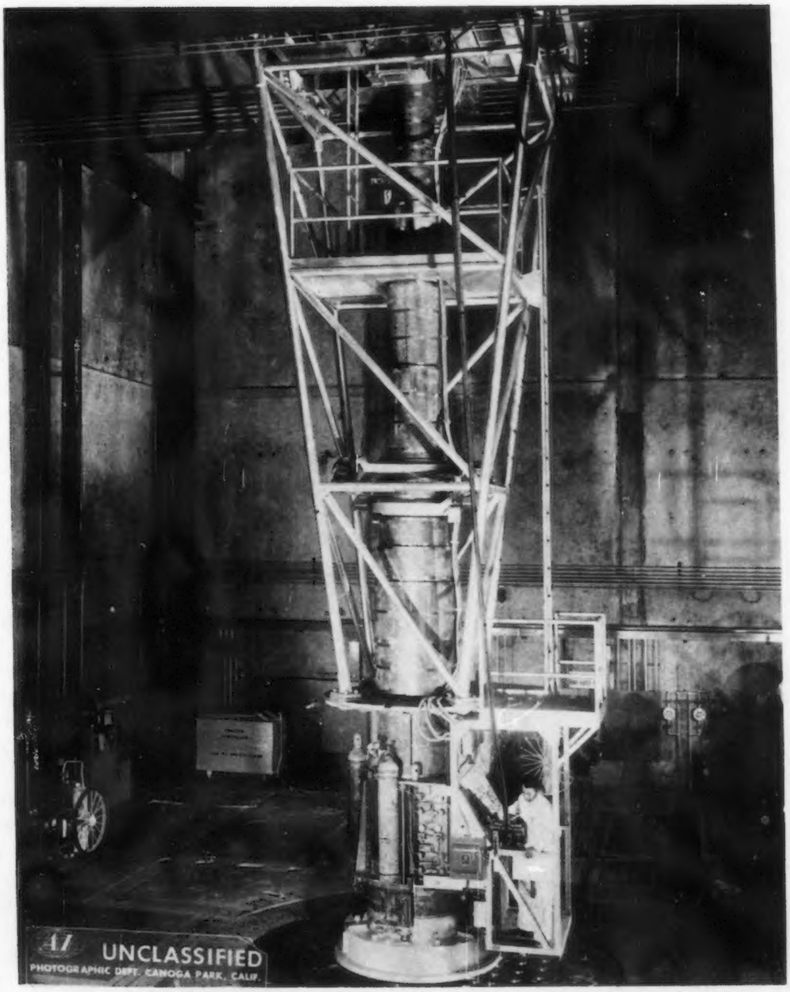


FIGURE NO. 4

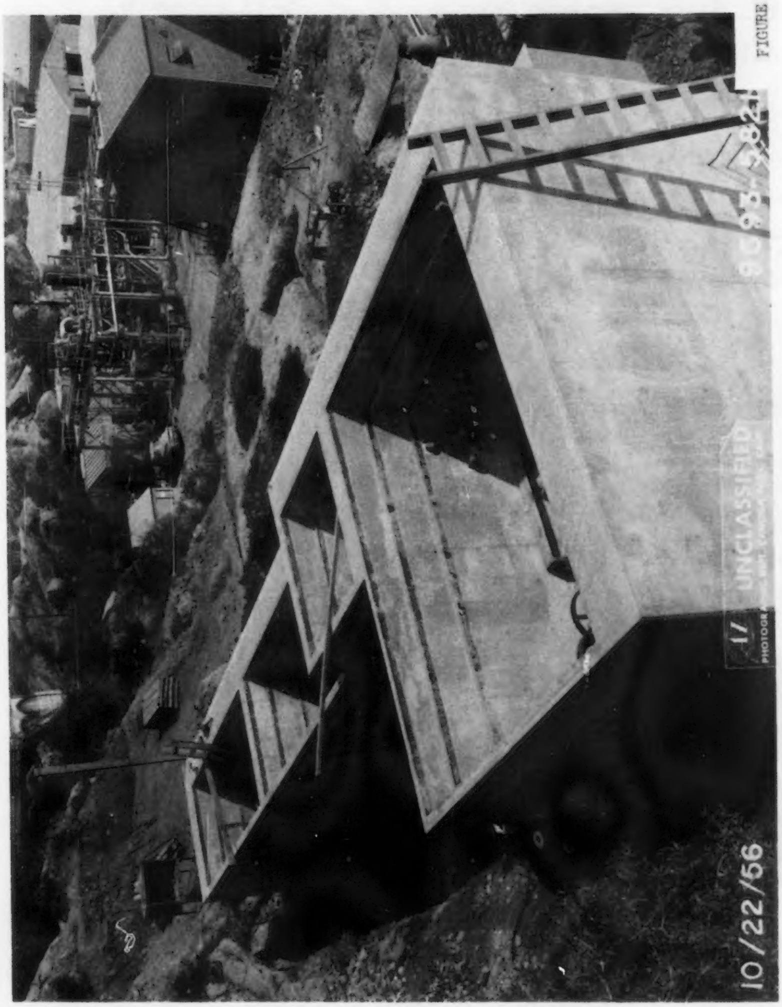


FIGURE NO. 5

Radioactive gas is pumped by means of a compressor into one of four "decay tanks". (Fig. No. 6) The gas is sampled to determine its activity and decay rate. When the gaseous activity has decayed to an acceptable level it is released to the atmosphere through the reactor building stack. The dilution rate is selected so as to reduce the activity below the maximum permissible concentration for beta-gamma emitters.

A continuous radiation monitor samples this exhaust gas and automatically closes the discharge valve if the activity gets above a preset value (3×10^{-6} uc. cc).

In practice, it has been observed that an extremely small quantity of active gas has been generated during the two years of reactor operation. The primary constituent has been sodium-24 which decays by a factor of 10 every two days and thus presents no release problems.

G. Thermal Considerations and Their Effects on Design

There are many thermal considerations which enter into the design of a nuclear power system. Several of those which went into the design of the SRE are listed below.

A thermal barrier is provided inside the reactor core tank to impose a uniform temperature gradient along the core tank wall. (Fig. No. 3)

In the design of the reactor core, provisions were made to equalize the radial expansion of the moderator cans because of the 400° F. axial temperature gradient. Design provided for the use of 304 stainless steel for the bottom support members and 410 stainless steel for the top. The smaller coefficient of expansion of the 400 series stainless steel reduces the radial expansion of the hotter portion of the cans and thus provides for more uniform fuel channel alignment as the core temperature gradient increases.

Because of the large change in density of sodium as a function of temperature, component elevations were selected to provide for natural convection sodium flow. This convection flow provides for reactor cooling after shutdown whether or not the coolant pumps are in operation, thus providing an inherent safety feature.

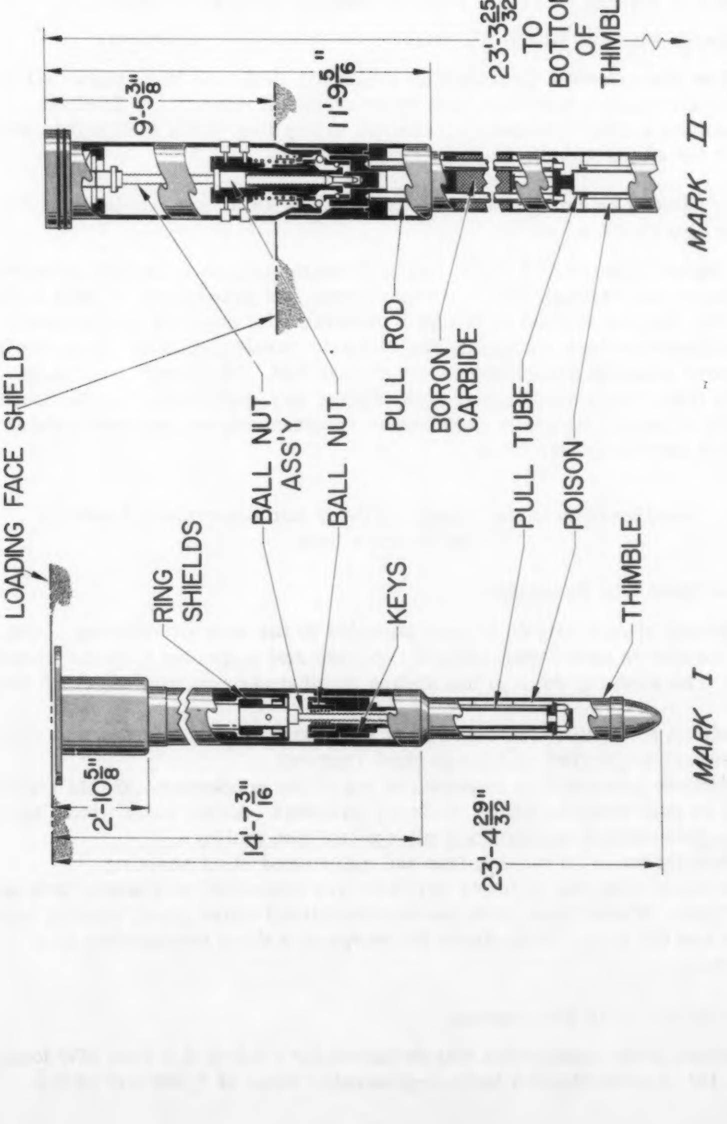
Control rods which contain boron nickel control elements are used to regulate the reactor power level. (Fig. No. 7) Boron is used because of its ability to absorb neutrons. Because of the neutron-alpha reaction the heat generation in the control elements is high. These elements must be cooled and yet retain complete freedom of motion. This is accomplished by holding a close tolerance between the control element and the thimble container which is cooled by circulating sodium. The gas anulus between the control element and the thimble is selected so as to provide for adequate heat transfer and yet allow for freedom of motion throughout the complete range of operating temperature.

4. Heat Production and Utilization

A. Necessity for Providing for Some Heat Disposal Under All Operating and Shutdown Conditions

During reactor operation, because of the fission process, radioactive isotopes are produced in the fuel. These isotopes, or fission products undergo radioactive decay even after the reactor shutdown. The radioactive decay

FIGURE NO. 6



9693- FIGURE NO. 7

process releases beta particles which are absorbed in the fuel material. These particles give up their kinetic energy of motion in the form of heat.

The power decay curve (Fig. No. 8) illustrates the rate of power decay after reactor shutdown. It is evident from this curve that some cooling should be provided after shutdown to remove this "afterglow" heat.

The SRE contains an auxiliary coolant system to provide for shutdown cooling. This auxiliary system is only a backup since normal afterglow heat is removed by the main coolant system.

B. Desire of Making Use of as Much of Reactor Heat as Possible

1. Early Plans to Waste All Heat

The experimental reactor was originally designed to dissipate all heat to the air through air blast heat exchangers. However, the Southern California Edison Company contracted to buy this waste heat and to utilize it for the production of electrical energy.

C. Desire to Get as Much Operating Experience With the Combination of Reactor and Turbine

The higher efficiencies of the modern steam stations have been attained for the most part through higher temperatures and pressures. Liquid sodium used as the reactor coolant in the SRE promises the elevated temperatures matching these modern stations. The ultimate result of this development will be the more efficient utilization of our atomic fuel. The experience to be gained by tying this experimental type reactor to a steam plant is the Southern California Edison Company's contribution to extend the experimental value of the project into the power field.

5. Consideration in the Design of the Steam Generating Facility Using Reactor Heat

A. Steam Plant Site Description

The steam station site is located adjacent to the reactor building. (Fig. No. 9) The site is paved with asphalt concrete and is graded to drain toward the east. The easterly edge of the station is located on approximately 6 feet of fill.

The steam generator, the deaerator, the steam jet air ejectors and associated piping are supported on braced steel frames.

The turbine generator is mounted on top of the condenser and this entire assembly is supported on belled concrete caissons. Minor foundations and the steam generator foundation are all spread type footings.

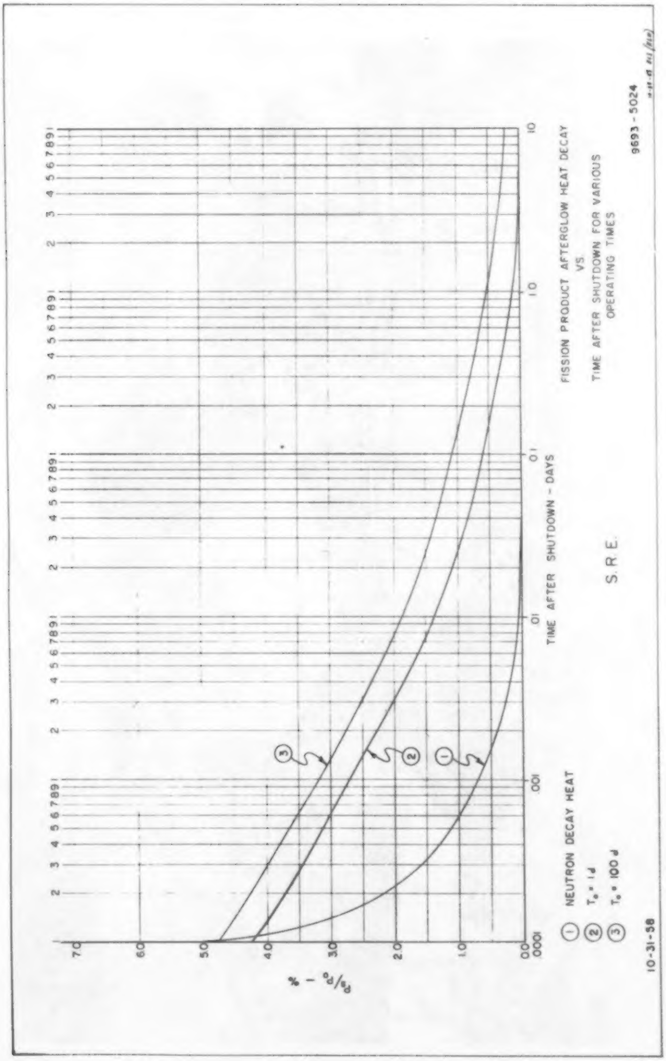
The control house is steel frame and galvanized steel sheeting.

Waste lines from the sanitary services are connected to a septic tank and leeching field. Waste lines from the demineralized water plant, cooling tower blowdown and the tower basin drain discharge to a ditch terminating in a settling basin.

B. Steam Plant Cycle Description

The steam plant installation was designed for a 6.0 to 6.5 Elec MW load. (Fig. No. 10) It consists of a turbine-generator rated at 7,500 KW with a

FIGURE NO. 8


FISSION PRODUCT AFTERGLOW HEAT DECAY
VS.
TIME AFTER SHUTDOWN FOR VARIOUS
OPERATING TIMES
S.R.E.
TIME AFTER SHUTDOWN - DAYS
Ps/Po - %
1 NEUTRON DECAY HEAT
2 To = 1d
3 To = 100 d
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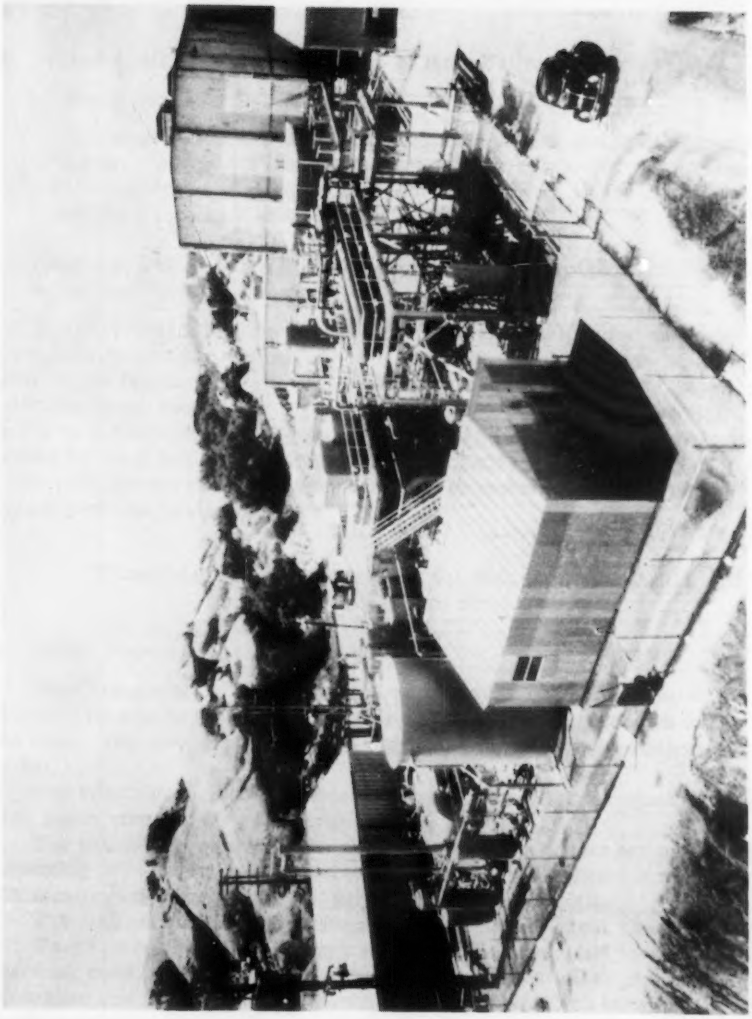
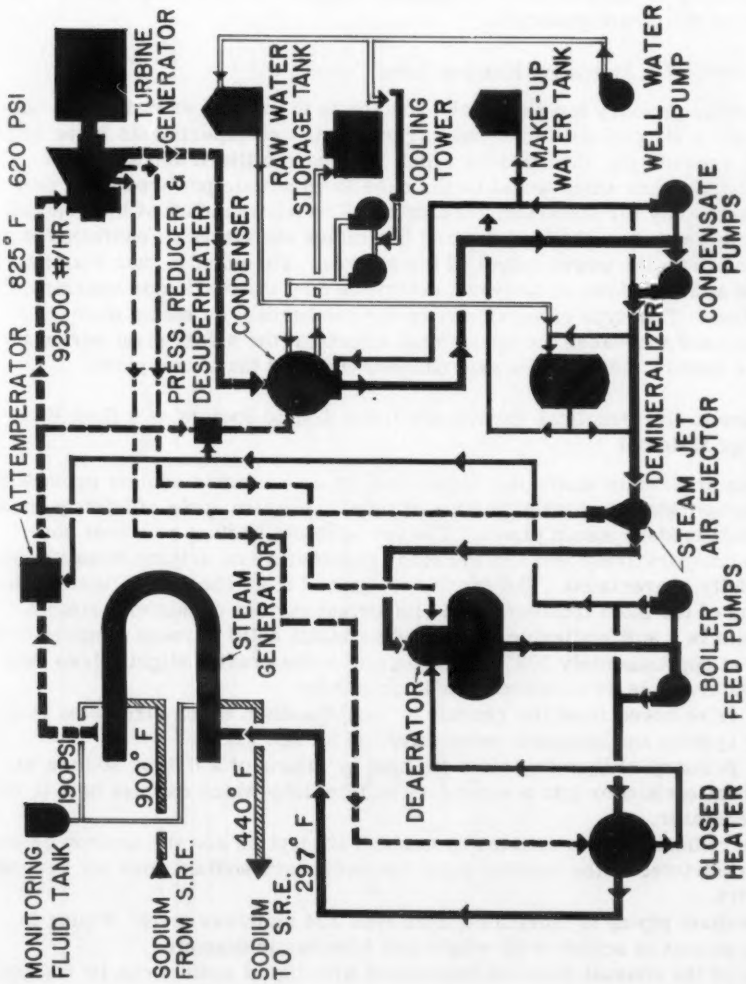


FIGURE NO. 9



BASIC FLOW DIAGRAM - STEAM PLANT

FIGURE NO. 10

liquid sodium to water through type steam generator. A second steam generator not shown on the schematic has been piped in parallel to the generator shown. This generator is a zone type exchanger utilizing a boiler drum and superheater section. This unit will be operated later in the experiment to determine its feasibility for this type operation.

Design steam conditions at the turbine throttle are 625 psig at 825° F and the extraction cycle includes two stages of feedwater heating. A mixed bed demineralizer is utilized in the condensate cycle to insure a high purity water supplied to the steam generator.

C. Necessity for Following Reactor Load Closely

Since the primary function of the reactor is to provide experimental data, its operation is contingent upon the various tests and experiments being conducted. Accordingly, the controls which have been utilized are arranged different from that which would be the case in an atomic powered plant designed primarily for electrical generation. The power output of the reactor is not dependent upon the load demand but rather the electric generation is adjusted to handle the power output of the reactor. The plant is thus started, operated and shut down at intervals determined by the particular experiment in progress. This type of service is by far the hardest on steam plant equipment and increases the operational efforts of the steam plant personnel. All these factors increase the experimental value of the steam plant.

D. Desire to Get Practical Experience Using Liquid Sodium as a Heat Exchange Material

It was previously stated that liquid sodium as a reactor coolant promised higher temperatures which give hope of reaching steam cycle efficiency equal to the most modern steam plants. The use of liquid sodium as a heat conveyer is comparatively new and the many considerations arising from its use are not fully appreciated. The experience gained from the use of liquid sodium is one of the most interesting and important factors of this experiment.

Sodium is a soft malleable silvery white metal solid at room temperature. It melts at approximately 208° F and has a specific gravity slightly less than that of water. It is an excellent conductor of heat.

Heat is removed from the reactor by liquid sodium which circulates in a primary system and becomes radioactive. (Fig. No. 11)

This primary sodium transfers its heat by means of a U-tube sodium to sodium heat exchanger into a secondary sodium loop which carries heat to the steam generator.

Since radioactivity is created by neutron absorption and the neutron generation is restricted to the reactor core, the secondary sodium does not become radioactive.

All sodium piping is fabricated from type 304 stainless steel. Piping in the main circuit is schedule 40 weight and 6 inches in diameter.

Some of the unusual features associated with liquid sodium can be realized by an examination of the circulating pumps. (Fig. No. 12)

The sodium pumps are modified hot process pumps mounted vertically. Frozen sodium seals are provided around the shaft and on the casing of the pumps. The seal is cooled with tetralin to solidify the sodium. The seals are blanketed from the atmosphere with helium gas to prevent oxidation.

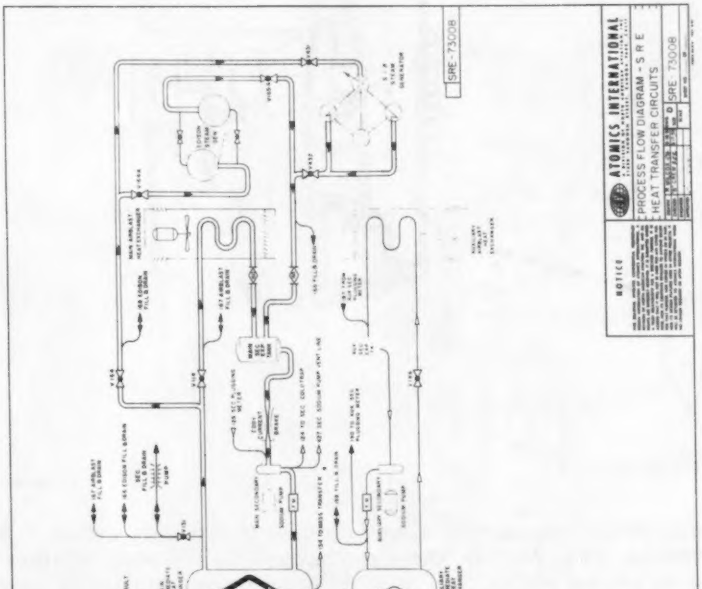

REACTOR
ATOMICS INTERNATIONAL
PROCESS FLOW DIAGRAM - S R E
HEAT TRANSFER CIRCUITS
SRE-73008

FIGURE NO. 11

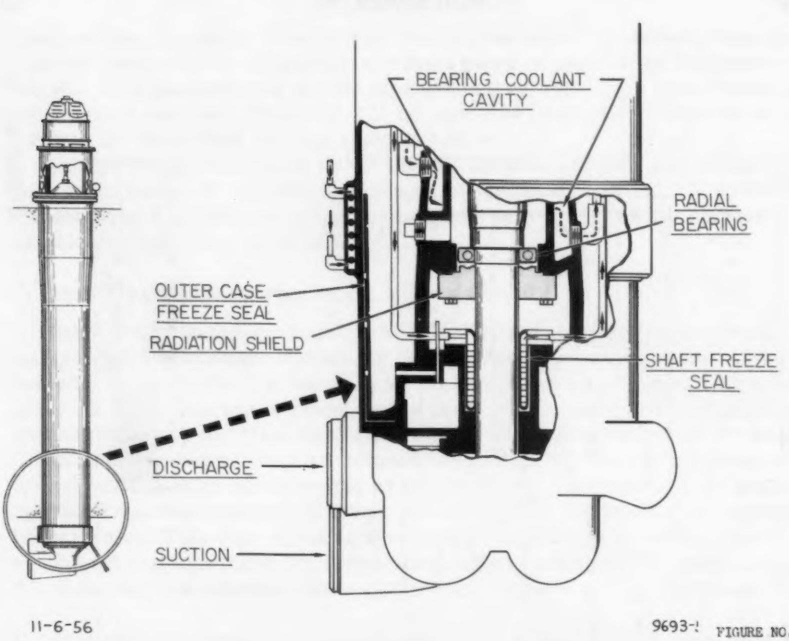


FIGURE NO. 12

Low capacity electromagnetic pumps are used to circulate sodium in the service systems. (Fig. No. 13) These pumps have the advantage of simplicity and contain no moving parts. They are not used for main circulating pumps because of their low efficiency.

All of the large sodium valves used in the primary and secondary sodium systems have the normal shaft packing replaced with a cooling jacket to provide a frozen sodium seal. (Fig. No. 14) All small valves used in the sodium system are of the packless bellows seal type.

6. Considerations in Connection With the Water Supply to the Station

A. Water Supply

The reactor and steam plant installation is supplied with well water.

Water is pumped to a 50,000 gallon storage tank located on a hill adjacent to the reactor and steam plant site.

B. Cooling Tower

The greatest water demand at this installation is that required by the cooling tower for cooling the water circulated through the condenser.

This induced type cooling tower is designed to handle 7000 gpm of circulating water.

Raw water make-up is supplied at the rate of from 50 to 150 gallons per minute when the plant is under full load.

The tower water is held to 5 concentrations by means of continuous blowdown.

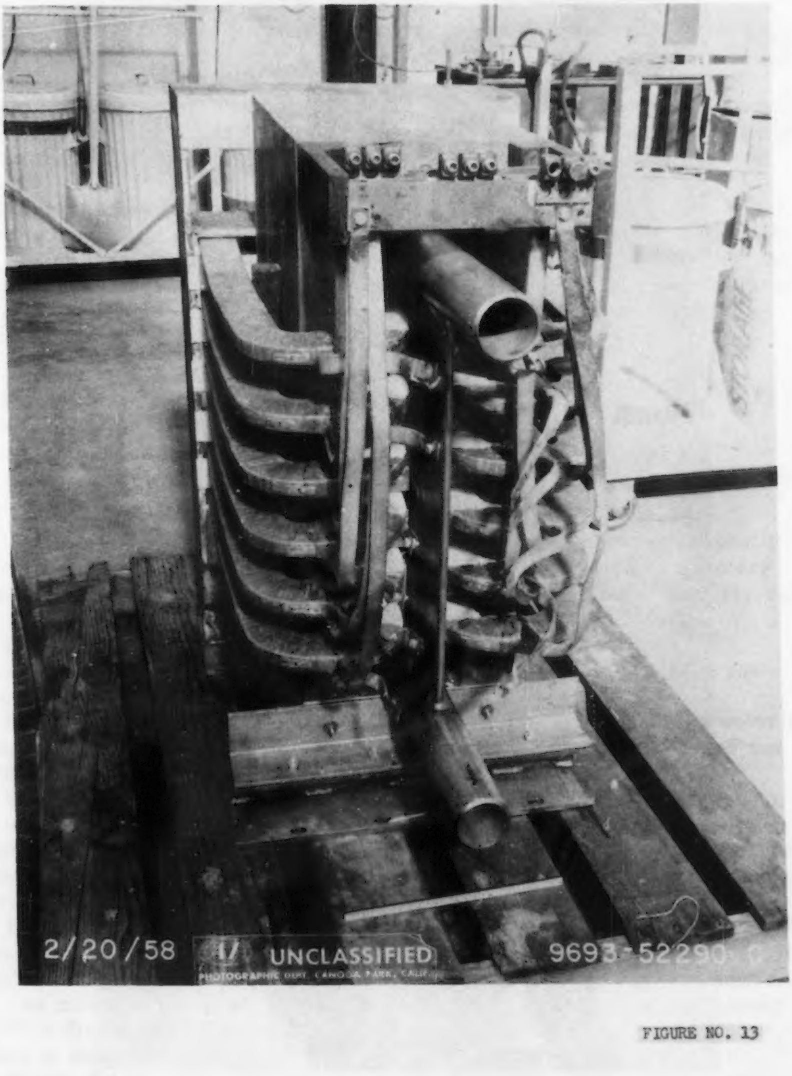


FIGURE NO. 13

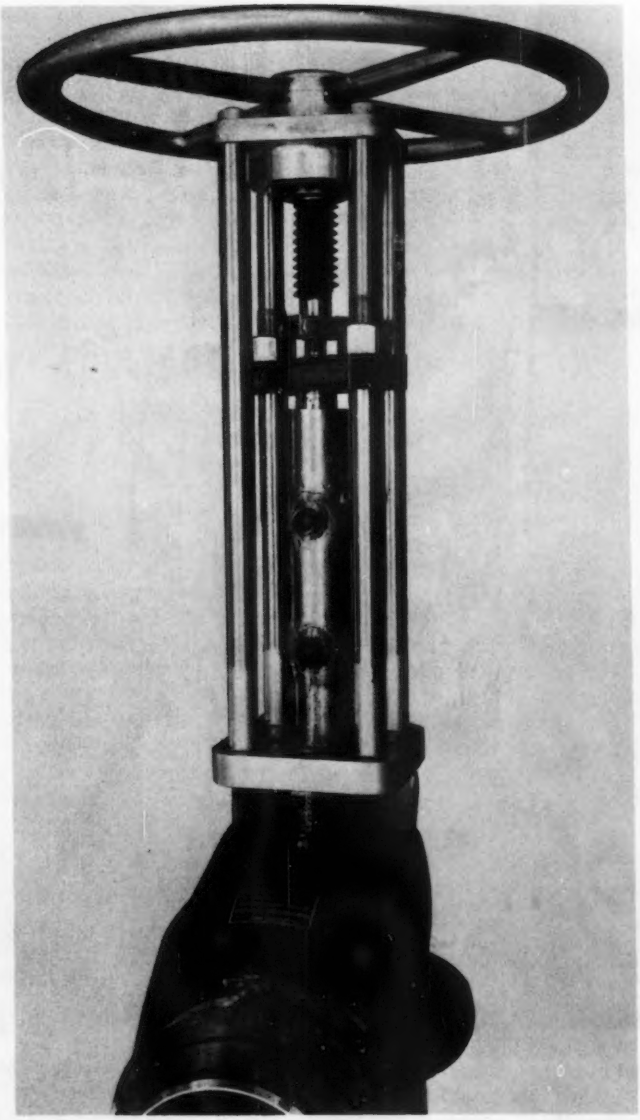


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FIGURE NO. 14

C. Requirement for High Purity Water

The steam generator now in operation at the plant is a U-type heat exchanger with sodium flowing on the shell side and water flowing on the tube side. (Fig. No. 15) The tubes are double walled with mercury in the annular space between the inner and outer tubes.

The mercury acts as a monitor to detect by a loss or gain of pressure a leak in either the low pressure sodium or high pressure water side.

The steam generator is preheated to 350° F before the introduction of sodium. This is accomplished by circulating heated water through the generator with one of the boiler feed pumps. A nitrogen pressurizing system keeps approximately 165 psig pressure on this heated water during the preheating period.

Due to the fact that the steam generator is the once through type in which water is heated and evaporated to complete dryness while passing through the unit, solids may either deposit on the heating surfaces or carry over into the turbine where they may form sludge on the turbine blades. The manufacturer has recommended for the prevention of objectionable solid deposition the following feedwater conditions be maintained:

1. pH 9.5 - 9.6
2. Fe 0.01 ppm (maximum)
3. Solids 0.5 ppm (maximum)
4. O_2 less than .005 ppm

D. Demineralizer

In order to keep the total solids below the limits indicated, an automatic mixed bed demineralizing plant has been provided to continuously or intermittently polish the condensate and process all raw water make-up. (Fig. No. 16)

Raw water entering the demineralizer plant passes through a graded anthracite and activated carbon filter. These filters remove solids in suspension and chlorine.

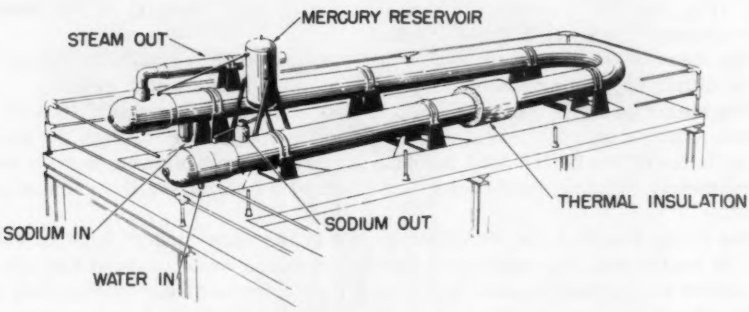
Following its discharge from the filters approximately 60% of the water is passed through a hydrogen zeolite cation exchanger. The filtered water and cation effluent are then blended before entering a vacuum deaerator.

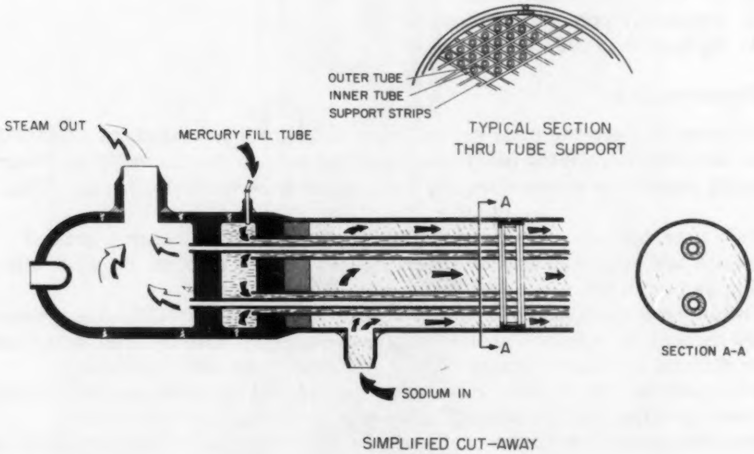
Water enters the deaerator tower at the top and flows downward through the tower packing into the storage tank below. Water vapor and noncondensable gases are drawn off through a pipe from the top of the tower and exhausted to the atmosphere through a high vacuum pump. Two booster pumps extract effluent from the storage tank.

Beyond the deaerator the partially treated water is blended with condensate. The blended water passes through two full-flow cotton filters which remove all suspended matter before entering the mixed bed ion exchangers.

The mixed bed units utilized cation and anion resin beads approximately 1/16" in diameter. These beads remove all impurities in the water except solids in suspension.

Tanks of 1500 gallon capacity for acid and caustic storage are provided for regeneration of cation and anion beds. The hydrogen zeolite cation exchanger is regenerated with acid and as a mono bed unit, however, the successful regeneration of the mixed bed units depend upon the separation of the cation and anion beads into strata prior to the simultaneous acid and caustic





Heat Exchanger - Construction

FIGURE NO. 15

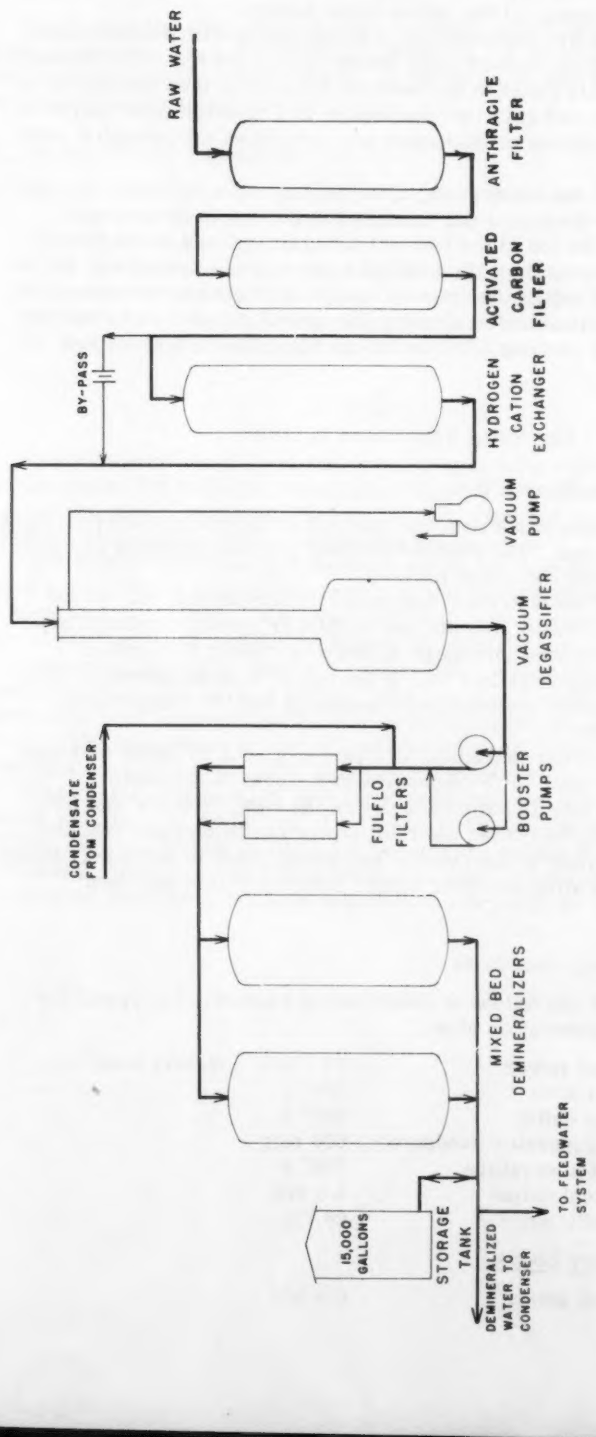

RAW WATER
ANTHRACITE FILTER
ACTIVATED CARBON FILTER
BY-PASS
HYDROGEN CATION EXCHANGER
VACUUM PUMP
VACUUM DEGASSIFIER
CONDENSATE FROM CONDENSER
BOOSTER PUMPS
FULFLO FILTERS
MIXED BED DEMINERALIZERS
15,000 GALLONS
STORAGE TANK
TO FEEDWATER SYSTEM
DEMINERALIZED WATER TO CONDENSER

FIGURE NO. 16

flushing. Strata separation is possible only after some cation capacity is exhausted resulting in a change of the cation resin density.

The mixed bed units are regenerated by backwashing with demineralized water. During this step the lighter anion beads rise to the top while the heavier exhausted cation beads settle to the bottom. Caustic is then introduced at the top of the exchanger and acid into the bottom in a simultaneous operation. The caustic and acid combine at the center and are drawn off through a pipe grid system.

The regeneration of the beads bring them back to equal density. In order to successfully re-mix the cation and anion beads the bed is flooded with enough water to cover the top of the bed and air is introduced at the bottom. This air agitation re-mixes the cation with the anion beads throughout the bed.

The regeneration of any of the units in this plant is push button automatic. That is, after an alarm is actuated alerting the operator that a unit requires regeneration, by simply pushing a button the operator starts the unit into its regeneration cycle.

7. Operating Experience to Date

A. Start of Reactor and Steam Plant

The experience to date has shown the start-up of the plant is very straightforward and uncomplicated. The reactor and heat transfer systems are preheated to 350° F by use of electrical heaters.

The reactor is then made critical and power slowly increased. As the sodium temperature increases, the normal feedwater system is placed in operation and boiler pressure increased to 600 psig. When the sodium temperature reaches approximately 460° F feedwater flow is started to hold the generator sodium outlet temperature constant at 460° F. Steam is bypassed to the condenser.

Reactor power is further increased at a rate to give a 50°/hour increase in sodium temperature until 70° F of superheated steam is available.

At this point the turbine is rolled and put on the line. Reactor and feedwater flow are gradually increased until full power conditions are reached.

The combined operation of the reactor and steam plant is extremely stable and requires very little attention after steady state operation has been achieved.

B. Full Power Operating Conditions

At 20.6 thermal MW the following conditions are found to be typical for the reactor and steam generation plant.

Thermal power	20.2 MW (primary loop)
Reactor inlet	539° F
Reactor outlet	908° F
Steam generator pressure	600 psig
Steam temperature	780° F
Electrical output	5.8 MW
Plant efficiency	28.7%
<u>Auxiliary Loops</u>	
Thermal power	0.4 MW

C. Steam Generator

Other than for some stratification troubles which have been corrected, the steam generator has performed satisfactorily. A thorough examination of the unit to determine the extent, if any, of stress corrosion and internal deterioration has not been attempted.

D. Water Control Experience

1. Water conditions at start up insofar as oxygen is concerned could be improved by pegging the deaerator i.e. supplying it with heat directly from the steam generator.
2. It has been found that polishing condensate once per day has been sufficient to hold the total solids down to acceptable limits.
3. The use of morpholine and hydrazine as a volatile water treatment method has been satisfactory.

8. CONCLUSION

In conclusion, it can be stated that the operating experience to date has demonstrated the technical feasibility of this installation. Furthermore, it has proven that maintenance can easily be performed on large sodium systems without hazard or complicated procedures.

The Southern California Edison Company has gained experience in operating a once-through type boiler requiring the highest standards of water purity control and treatment. This knowledge will have a direct application in the use of similar equipment in the Edison Company's newest steam stations.

Biological shielding required around this reactor or any other type power reactor is one of the major factors affecting cost. Although no unusual or entirely new civil engineering techniques or procedures were developed on this station, the experience gained will aid the civil engineer in his problem of lowering the cost of shielding in future projects.

The overall information thus far gained has been used to advantage by North American in the design of a 75 Elec. MW sodium reactor known as the Hallam Nuclear Power Facility for Consumers Public Power District in Nebraska. With this new full scale power plant in operation the design techniques, operating experience and economics will be more fully understood.

Journal of the

POWER DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION

(Over)

ROCKFILL DAMS: CHERRY VALLEY CENTRAL CORE DAM[a]

Discussion by F. L. Lawton

F. L. LAWTON,[1] M. ASCE.—The authors are to be congratulated on their comprehensive treatment of design, construction and operational features of the 330 feet high central-core Cherry Valley Dam. The basic justification, the economic one, for the selection of the central-core design is clearly given, in the cost of the compacted decomposed granite in place in the core at about one-half the cost of freshly-quarried granite in place in the shoulders.

It would add to the value of the paper if the authors would explain the justification for the use of an earthquake allowance at 0.05g in the analysis of stability of the rock slopes, in view of seismic activity in the general area.

The settlement of the core at the 323 feet high maximum section, at station 16+00, over a two-year period amounts to 0.45 feet or 0.14% of the embankment height. On the other hand settlement of the upstream berm at elevation 4580 amounts to about 2 feet. Even though the rockfill was placed in relatively low lifts of 30 feet, it seems to have been well sluiced. Consequently the differential settlement appears to have been much more significant than the shallow cracks between the core and the transition zone would indicate. Could this have been due to the manner in which transition material was placed during the summer of 1954?

The authors' observations on deflection of the dam under load and the substantial rebound when the reservoir was drawn down are most valuable and unusual. The net upstream movement of the several stations at which measurements were made, except at station 16+00, is indeed hard to explain. Certainly the hypothesis of unequal settlement between the upstream and downstream portions of the embankment causing a net movement in an upstream direction at the crest of the dam is difficult of acceptance. The behaviour of the dam as an imperfectly elastic structure under horizontal loading is also rather unusual in view of the nature of the core material, described as being 95% silty sand, a rather natural result of its origin by deep chemical weathering of granitic bedrock.

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a. Proc. Paper 1733, August, 1958, by H. E. Lloyd, O. L. Moore and W. F. Getts.

1. Chf. Engr., Power Dept., Aluminium Labs. Ltd., Montreal, Canada.

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ROCKFILL DAMS: BROWNLEE SLOPING CORE DAM[a]

Discussions by F. L. Lawton, Huai-yun Hsu and Reijo Ito

F. L. LAWTON,[1] M. ASCE.—The author has established as criteria applied to the final design of the Brownlee dam those considerations which enter into the economic design of any rockfill dam. Two features are of particular note in this connection: the thin impervious core necessitated by the absence of large volumes of suitable soil, and the deep overburden in the river channel. Another aspect of major importance in securing optimum economy at Brownlee, in common with many other rockfill dams, was the achievement of a close balance between rock excavation and rockfill requirements.

Effective utilization of geological features usually results in superior economy. This would appear to have been fully realized in the use of the rock nose in the left abutment, of the overburden in the river bed and of dyke No. 4 for a portion of the core trench.

The method used to facilitate placement of the clay core in the deepest portion of the core trench, despite seepage, was ingenious. It is a method analogous to the use of a concrete pad on a jagged, steeply-dipping bedded formation in order to use tractor-drawn compaction equipment in the relatively-restricted area in the lower part of a river channel, as at Kenney dam,(1) and also achieve better grout distribution.

It is rather difficult to fully appreciate how ". . . special drilled and grouted holes near the left abutment" deterred some of the artesian water from entering the core trench in view of the geological profile along the centre line of the core trench. Perhaps the author could amplify his observations on this point.

The overtopping of the incomplete dam during the period from February 24, 1957, to some time about mid-June through a gap about 250 feet wide at approximately Elevation 1809 provided an extremely valuable experience since an estimated 40,000 to 50,000 cfs flowed over the fill. The author's observations on the behaviour of the rockfill and impervious core would have been much more valuable if the effect on the fine and coarse filter zones had been noted. Information on the size of the rockfill would also be useful.

Under conclusions, the author states the "unit cost for rock placed in the dam is estimated to be about half the cost for similar construction 10 to 15 years ago". This is such an important factor in the choice of type of dam for a given location, it is to be hoped the author can justify the conclusion by giving supporting figures.

In connection with the construction of the Brownlee dam on the consolidated alluvials (sand, gravel and boulders), it is worth noting the left-hand bank

a. Proc. Paper 1734, August, 1958, by Torald Mundal.

1. Chf. Engr., Power Dept., Aluminium Labs. Ltd., Montreal, Canada.

section of the Bersimis dam(2) is built on a maximum thickness of about 300 feet of a succession of till and varved clay and silt strata, of glacial origin. This material is obviously much finer than that underlying the Brownlee dam. Both Brownlee and Bersimis dams are striking examples of one of the attractive economic features of the rockfill dam, its application to apparently difficult, if not impossible, dam sites where the properties of well-designed thin impervious cores, filters and blankets are fully utilized.

The author's conclusion on the feasibility of passing flood water over a partially completed rockfill dam is an important one, as such a procedure can be utilized to minimize diversion costs, where probable flood flows to be handled during construction can be reasonably closely evaluated and construction schedules permit overtopping.

REFERENCES

1. "The Kenney Dam". Harry Jomini. The Engineering Journal, November, 1954, Vol. 37, No. 11, p. 1386-1397.

2. "Rockfill Dams: The Bersimis Sloping Core Dams". F. W. Patterson and W. H. MacDonald. Proc. Paper 1740. Journal of the Power Division, ASCE, August, 1958.

HUAI-YUN HSU,[1] M. ASCE.—The stability of Brownlee rockfill dam was briefly discussed in Mr. Mundal's paper. The detail analysis which follows may be of interest to design engineers.

Upstream Slope

In rockfill dams, generally, the upstream rockfill has better shear strength than the filter zones and the filter zones better than the impervious core. Unless the upstream rockfill material contains too high a percentage of fines or clay, the weakest shear plane should be in the impervious core. In Brownlee, the critical surface selected for stability study is along ABCD as shown in Fig. 1. The vertical line CE divides the wedge into two parts—Block ABCE sliding down along the potential failure plane BC and Block ECD resisting the failure by its shearing strength along the base CD. The position of line CD is so chosen that a minimum factor of safety will be obtained. The exact location of AB is immaterial, as the forces acting on it are relatively small and, therefore, neglected.

Taking block ABCE as a free body, besides the resisting force R (equal to $W_1 \tan \phi_R$), we have the forces T and N, tangent and normal to line CD, to keep the weight W_2 in equilibrium. T and N are solved by the force polygon.

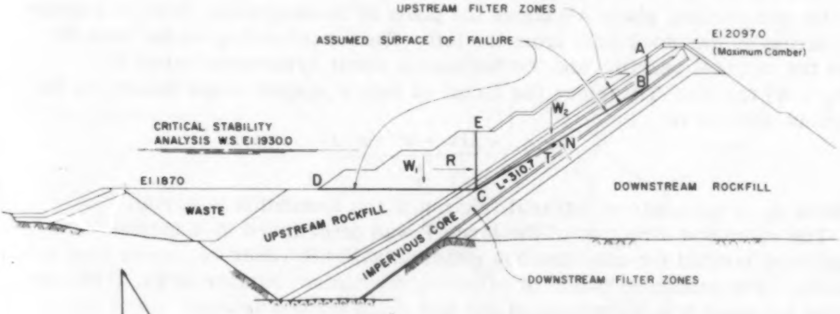
Since the potential shearing strength, P, along the line CD is equal to $N \tan \phi_C + CL$, the factor of safety against failure along CD is P over T. However, in computing the factor of safety for the entire surface ABCD, the value of R should be reduced by the factor of safety before entering the force polygon.

With an internal friction angle of 40 degrees for the upstream rockfill, the factor of safety for the upstream slope at Brownlee was found to be 1.56 for

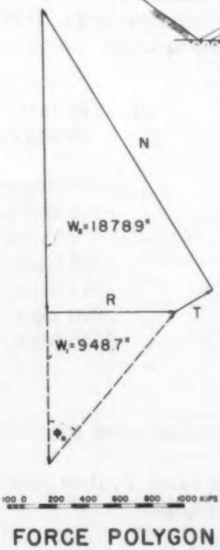
1. Project Engr., International Eng. Co., Inc., San Francisco, Calif.

SOIL CHARACTERISTICS

PROPERTY	UNIT	ROCKFILL	IMPERVIOUS CORE
MOIST WEIGHT	PCF	115	110
SATURATED WEIGHT	PCF	134.4	117.4
SUBMERGED WEIGHT	PCF	72	55
COHESION, C	PSF	—	1000
ANGLE OF FRICTION, $\phi°$	Deg	40°	15°



MAXIMUM SECTION



FORCE POLYGON

ASSUMED CONDITIONS INITIAL FILLING OF RESERVOIR
WITH CRITICAL STABILITY ANALYSIS W.S. AT El 1930.0
ASSUMED SURFACE OF FAILURE ABCD

ASSUMED FACTOR OF SAFETY OF ROCK (FS_R)	1.0	1.5	2.0
W_1 =	948.7^K	948.7^K	948.7^K
$R = \frac{W_1 \tan \Phi_R}{FS_R}$	796.0^K	530.7^K	398.0^K
$P = N \tan \Phi_c + CL$			
N (Scaled)	2,024^K	1,889^K	1,820^K
$N \tan \Phi_c$	542.3^K	506.2^K	487.7^K
CL	310.7^K	310.7^K	310.7^K
TOTAL P	853.0^K	816.9^K	798.4^K
T (Scaled)	272^K	500^K	614^K
FACTOR OF SAFETY OF CORE $FS_c = \frac{P}{T}$	3.14	1.63	1.30
FACTOR OF SAFETY OF FAILURE SURFACE ABCD		1.56	

CALCULATION OF FACTOR OF SAFETY

FIGURE 1
BROWNLEE PROJECT
STABILITY ANALYSIS
UPSTREAM SLOPE

the most critical condition of initial filling of the reservoir with water surface at El. 1930. Using 28 degrees for ϕ_R results in a factor of safety of 1.38 for the same conditions.

Downstream Slope

Brownlee rockfill dam is underlain by more than 100 feet of riverbed sand and gravel. Although dense and compacted, the riverbed foundation material has a smaller angle of internal friction than the main rockfill for the dam. The stability of the downstream slope was, therefore, analyzed along the contact surface, between the foundation and the dam, at different elevations across the river. The example shown in Fig. 2 is for El. 1700.

On any vertical plane AB above the plane of investigation, there is a force P, acting on the block ABC from the left. The forces acting on the base BC are the vertical reaction and the horizontal shear resistance (equal to $(P_V + W) \tan \phi_f$). Therefore the factor of safety against shear failure on the contact surface is:

$$F.S. = \frac{(P_V + W) \tan \phi_f}{P_H}$$

Where ϕ_f is the angle of internal friction of the foundation material.

The value and direction of the force P was determined by Engesser's graphical method for cohesionless material with the reservoir water load included. The minimum factor of safety on the contact surface at El. 1700 was found for point B at a distance of 280 feet from the downstream toe of the rockfill. The data and the result are as follows:

Critical tailwater elevation (max. T.W.)	El. 1827 feet
Angle of internal friction for rockfill, ϕ_R	40 degrees
Angle of internal friction for foundation material, ϕ_f	39 degrees
Moist weight of rockfill	115 lb/cu. ft.
Submerged weight of rockfill	72 lb/cu. ft.
P (scaled from Engesser's Envelope)	855 kips
P_V	515 kips
P_H	690 kips
W	2155 kips

$$F.S. = \frac{(515+2155) \tan 39^\circ}{690} = 3.1$$

Laboratory tests indicate that the values of the angle of internal friction used in the stability studies are conservative.

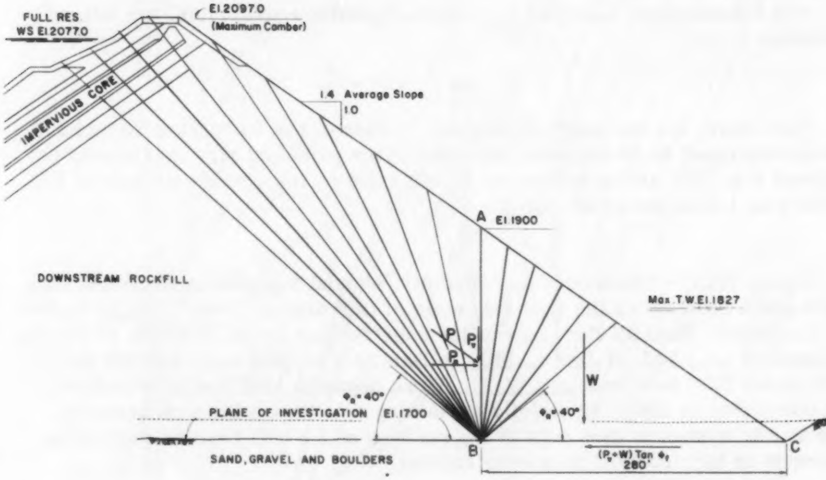
By utilizing Engesser's method of determining the value P, it is possible to determine the stresses at any point in the main rockfill.

$$\text{Vertical pressure, } P_v = \frac{d\,(W+P_V)}{dx},$$

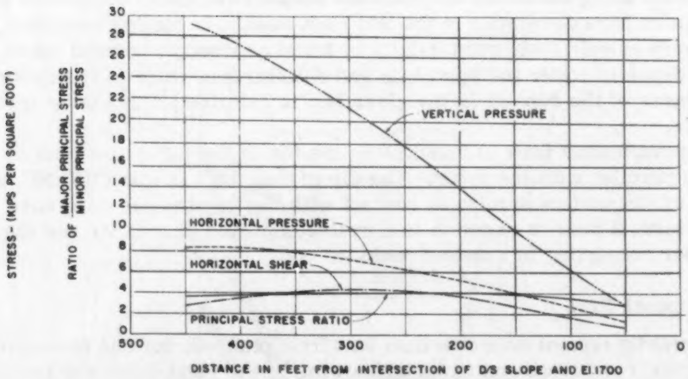
$$\text{Horizontal pressure, } P_h = \frac{d\,P_H}{dy},$$

$$\text{Horizontal shear, } \tau = \frac{d\,P_H}{dx},$$

where x and y are the horizontal and vertical axes. Graphical differentiation can be performed easily.



MAXIMUM SECTION



FOUNDATION STRESSES AT EL. 1700

FIGURE 2
BROWNLEE PROJECT
STABILITY ANALYSIS
DOWNSTREAM SLOPE

With vertical and horizontal stresses known, the major (σ_1) and minor (σ_3) principal stresses can be determined readily by Mohr's circle, or calculated by formulas. The stress distribution on the Brownlee dam foundation at El. 1700 is shown in Fig. 2 for an angle of internal friction of 40 degrees for the rockfill material.

For cohesionless material in a state of plastic equilibrium, the following relation holds:

$$\frac{\sigma_1}{\sigma_3} = \tan^2 (45° + \frac{\phi}{2})$$

Therefore, for the angle of internal friction of the foundation material at Brownlee equal to 39 degrees, the ratio of the principal stresses should not exceed 4.4. The actual maximum stress ratio on the contact surface at El. 1700 was 4.0 as shown on Fig. 2.

REIJO ITO.[1]—The writer has read Mr. Mundal's paper on Brownlee Dam with much interest as the type and scale of this dam are very similar to those of the Miboro Rockfill Dam now under construction by the Electric Power Development Co., Ltd. of Japan. Miboro Dam is a sloping core rockfill dam of 130 meter (426 feet) height (Fig. 1). Work began in 1957 and is scheduled to be completed in 1960. By briefly explaining the Miboro Dam construction, the writer wishes to give data to the readers which will become part of the records on high rockfill dam construction.

Geology

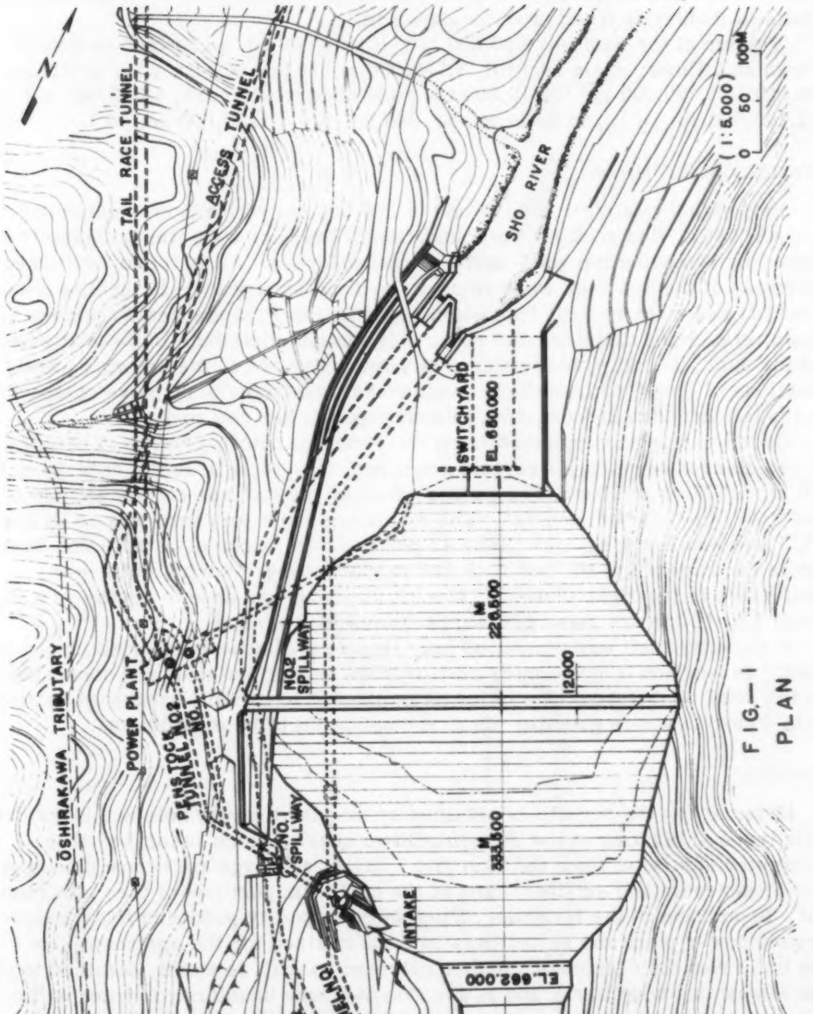
The bed rock in the dam site consists of quartz porphyry and granite porphyry. The bed rock of the right abutment is rather poor as talus deposits are found along the lower slope on the porphyritic rocks mentioned above and as cracks have developed in the bed rock itself. In the left abutment, quartz porphyry of relatively good quality is found exposed at several spots, but thick talus deposits cover the upstream and downstream sides of the abutment. Thickness of the deposit in the river bed is relatively thin and is less than 8 meters.

A conspicuous fault is found along the toe in the right abutment of the dam and is parallel with the river. The dip of this fault is about 70° NE, and the width of the portion coming in contact with the impervious core zone is about 10 meters, 2 meters of which is a well-compacted clay layer and the remainder consisting of crushed rock.

Selection of Type

A gravity type of concrete dam was first planned, but due to unfavorable geological conditions and especially owing to the right abutment fault, it was found that the volume of foundation excavation and concrete would have to be increased. Foundation treatments would also become more involved. Moreover, it was found that rock of good quality and materials suitable for an impervious core zone could be found in areas not very far from the dam site. For these reasons, a rockfill dam design was selected. A sloping core type

1. Chief of Construction, Miboro Dam Project, Electric Power Development Company of Japan, Miboro, Japan.


TAIL RACE TUNNEL
ACCESS TUNNEL
SHO RIVER
(1:5,000)
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SWITCHYARD
EL. 650.000
M
226,500
12,000
NO.2 SPILLWAY
POWER PLANT
PENSTOCK TUNNEL NO.2
NO.1
ŌSHIRAKAWA TRIBUTARY
M
333,500
NO.1 SPILLWAY
INTAKE
EL. 662.000
DIVERSION TUNNEL NO.1
DIVERSION TUNNEL NO.2
M
925
900
875
850
825
800
700
725
750
775
800

FIG.—1 PLAN

of rockfill dam, as shown in Fig. 2, was designed based on investigation results of materials found near the dam site.

Volume of the dam is 7,950,000 m^3 (10,400,000 cu. yd.) which is broken down as follows: main rockfill, 3,720,000 m^3 (4,880,000 cu. yd.); upstream rockfill, 1,760,000 m^3 (2,310,000 cu. yd.); impervious core, 1,630,000 m^3 (2,140,000 cu. yd.); and filter zones, 840,000 m^3 (1,100,000 cu. yd.).

Foundation Treatment

Excavation totaling 1,500,000 m^3 (1,960,000 cu. yd.) was completed during August 1958. Bed rock for the foundation of the core zone was completely exposed by removing top soil, sand and gravel layers. All soft and weathered portions of the bed rock were removed by hand excavation. In the upstream rockfill and main rockfill foundation areas it was first planned to remove silt, sand, gravel and disintegrated rock, which might contribute to the settlement of the dam. Actually, both abutments were excavated to the bed rock, while portions of the river bed where sand, gravel and rocks were more compact were left as the dam foundation without exposing the bed rock.

Grouting was accomplished from two concrete cutoffs, Fig. 2. There are three classes of holes for curtain grouting: "a"—10 m in depth, "b"—20 m to 30 m, "c" 30 m to 60 m. The packer grouting method was chiefly applied in the order of "c", "b", and "a". The maximum grout pressure was 15 kg/cm^2 (213 psi) and 2 to 3 kg/cm^2 (28 to 43 psi) near the surface. Brittle and weak spots encountered in the bed rock and in places where spring water was noticeable in the right abutment, blanket grouting was done to depths ranging from 7 to 12 meters using pressures from 2 to 3 kg/cm^2.

A vertical shaft was excavated near the crushed zone in the right abutment fault. In addition to grouting from four adits cutting the fault and connected to the shaft on different elevations, it is planned to backfill excavated portions with concrete where grouting alone is considered to be insufficient.

Rockfill

Material for the rockfill is obtained by means of coyote hole and large drill hole types of blasting in the Fukushimadani quarry which is located about 2 kilometer upstream from the dam site. Relatively large rocks are placed in the downstream and smaller sizes in the upstream portions of the main rockfill. In addition to the foregoing, muck of good quality produced from underground power plant and tunnel excavation is utilized for the upstream side of the main rockfill. Material for the upstream rockfill is mostly obtained from the Fukushimadani quarry and in addition oversize boulders produced at the filter materials plant from river gravel are also utilized.

In order to prevent segregation in sizes, the height of lifts of dumped rocks is limited to 4 meters in the upstream and 8 meters in the downstream portions of the main rockfill. A maximum lift height of 4 meters for the upstream rockfill zone is specified. All material placed in the main rockfill is sluiced and the amount of water applied is more than three times as much as the volume of the rock sluiced. The water pressure at the sluicing nozzles is approximately 5 kg/cm^2 (72 psi). No sluicing is required for the upstream rockfill.

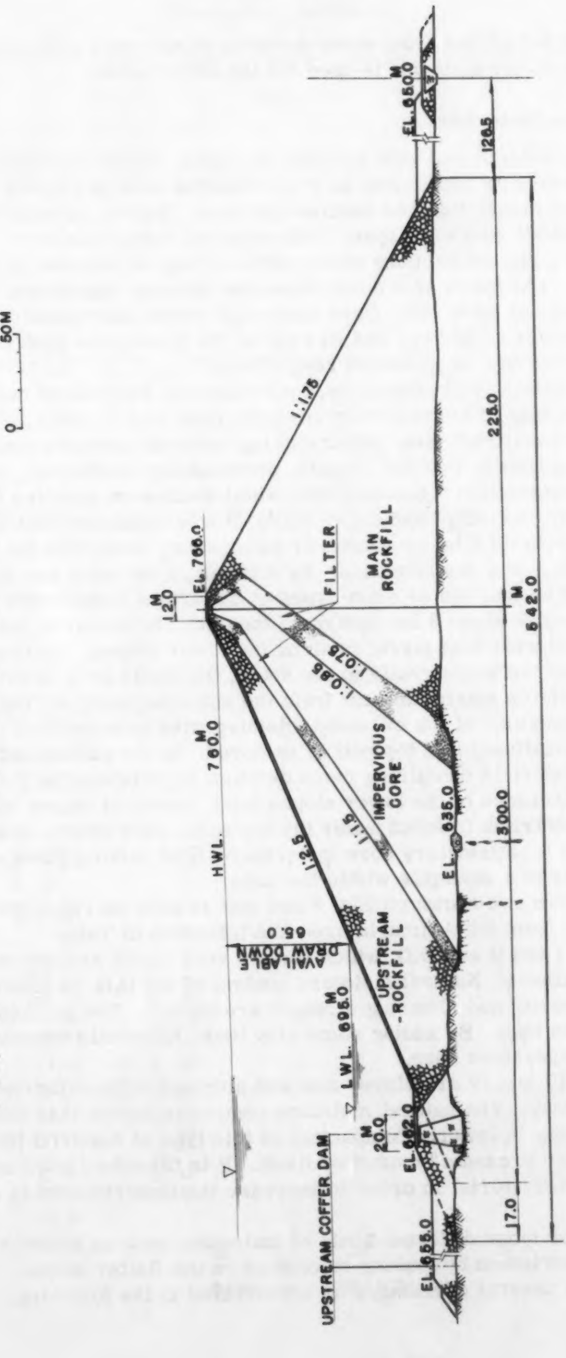

FIG.—2 TYPICAL SECTION OF DAM

Filter

Pit run sand and gravel from river deposits is screened at a special plant and all 200 mm minus material is used for the filter zones.

Impervious Core Materials

Due to heavy rainfall and high humidity in Japan, clayey materials having a suitable permeability coefficient as core material have in general a natural moisture content higher than the desired optimum. Miboro is situated in one of the heavy rainfall zones of Japan. Following the rainy season in June there may be several typhoons bringing heavy rains during the months of August and September. The job is shut down December through March due to considerable amounts of snow fall. Core materials which were found in the Miboro area similar to BI #148 and BI #151 at the Brownless Dam contained more than 60% and 30% of moisture respectively.

Under these unfavorable conditions, investigations were made covering wide areas extending 12 km upstream from the dam site in order to locate satisfactory core material pits. After making tests on moisture content, gradation, Atterberg limits, Proctor density, permeability coefficient, shearing strength and consolidation together with careful studies on samples obtained from various test pits, adits and auger holes, it was concluded that it would be impossible to obtain a large volume of satisfactory materials for the impervious core zone in a dry condition. As a result, a decision was made to adopt a policy of mixing two or more types of materials found in the Akimachi area which is located about 3 km upstream the dam. In this area the hill sides face the south and west with terraces along the lower slopes. As these terraces are wide and slope gently to the south, the whole area is favorably oriented to absorb the maximum heat from the sun effectively drying the material. In most portions of the hillsides, disintegrated granite (D.G.) is exposed when the relatively thin top soil is removed. In the gullies and lower slopes, clayey materials containing much detritus lie between the D.G. and the top soil. In the terraces of the lower slopes thick layers of clayey material containing much detritus is found under the top soil. As a result, it is much easier to prepare a satisfactory core material by first mixing these several types of materials in a stockpile within the area.

Gradation curves are shown on Fig. 3 and test results on representative samples obtained from the Akimachi area are tabulated in Table 1.

Test samples I and II are D.G. which contain very small amounts of fines and show no stickiness. Natural moisture content of the D.G. is lower than optimum while density and shearing strength are higher. The permeability coefficient is quite high. By adding some clay these materials become very suitable for an impervious core.

Test samples III and IV are clayey soil and although contain detritus, their fines are very sticky. The natural moisture content is higher than the optimum and the density is lower. Compaction of this type of material by rollers is very difficult and it cannot be used by itself. It is therefore used as a mixing agent with D.G. material in order to increase the imperviousness of the latter.

Mixing of two or more different kinds of materials such as mentioned above is successfully carried on by making stockpiles on the flatter slopes. In this method, there are several advantages as enumerated in the following:

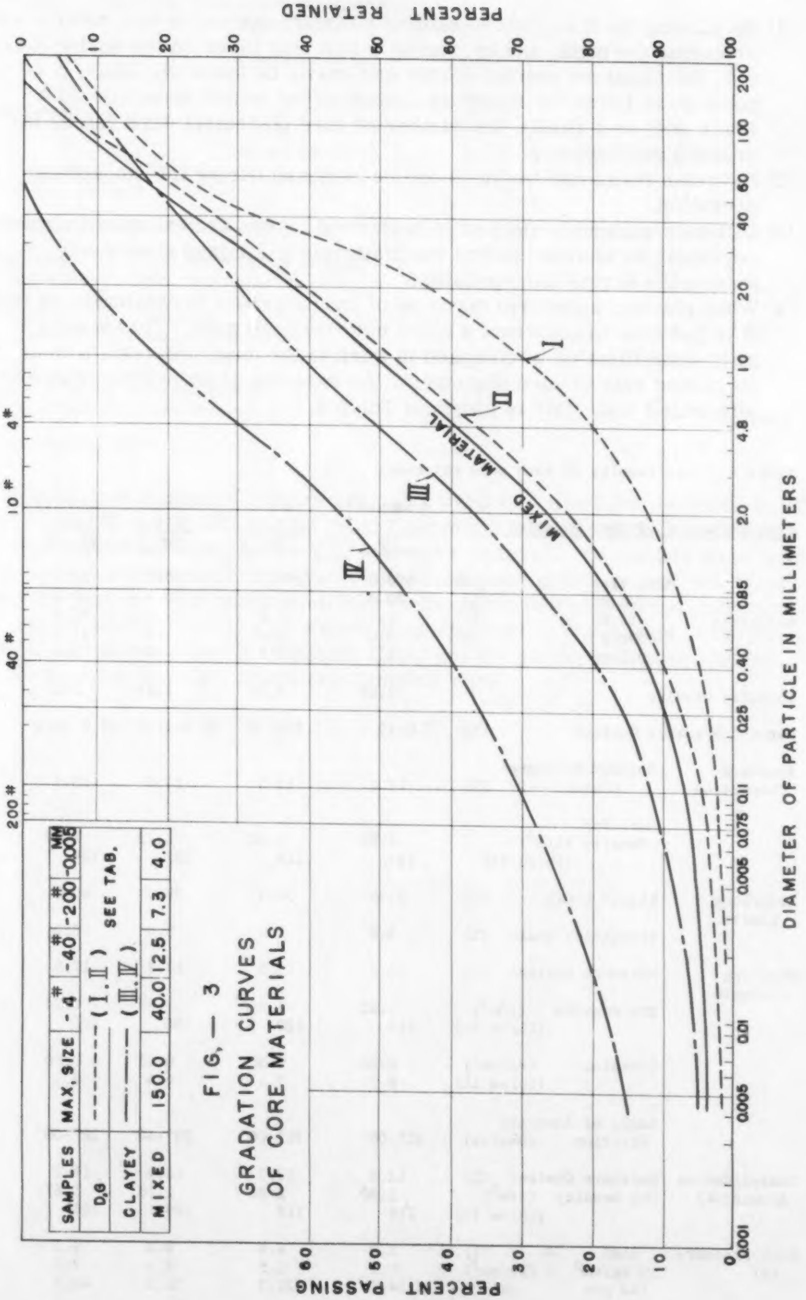

FIG. — 3
GRADATION CURVES OF CORE MATERIALS

(1) By placing the D.G., whose natural moisture content is low, after a rain or during the night, and by placing a thin clay layer on top during a dry day, the moisture content of clay can easily be lowered. Thus, it is possible to lower the moisture content of the mixed materials as a whole and, as a result, the number of core placement days can be increased considerably.

(2) Oversize rocks and boulders can be removed during the stockpiling operation.

(3) Different materials stocked in horizontal layers are excavated almost vertically by shovels so that the materials are mixed evenly and thoroughly during this operation.

(4) When placing, a uniform material of known quality is obtainable so that it is possible to construct a more uniform core zone. This also permits simplification of compaction work in the dam, and results in an increased rate of core placement. An example of properties of stockpile mixed materials is shown in Table 2.

Table 1. Test results on each core material.

Classification of Core Material			Sample I GW	Sample II GW	Sample III GM	Sample IV SM - SC
Gradation	Max. size	(mm)	400.0	200.0	300.0	50.0
	-4 #	(%)	20.0	37.0	50.0	70.0
	-40 #	(%)	6.0	10.0	19.0	40.0
	-200 #	(%)	3.0	4.0	11.0	27.0
	-0.005 mm	(%)	1.5	1.5	5.0	16.0
Specific Gravity			2.69	2.70	2.64	2.65
Natural Moisture Content		(%)	7.0-12.0	7.0-14.0	15.0-21.0	17.0-30.0
Standard Compaction	Optimum Moisture Content	(%)	12.5	13.5	17.8	18.5
	Max. Dry Density	(t/m^3)	1.95	1.90	1.72	1.70
		(lb/cu.ft)	122	119	107	106
Atterberg Limits	Liquid Limit	(%)	31.0	34.0	39.0	40.0
	Plasticity Index	(%)	3.0	5.0	5.0	10.0
Shearing Strength	Moisture Content	(%)	12.5	13.0	17.8	17.5
	Dry Density	(t/m^3)	1.83	1.92	1.69	1.70
		(lb/cu ft)	114	120	106	106
	Cohesion	(kg/cm^2)	0.60	0.50	0.45	0.50
		(lb/sq in)	8.5	7.1	6.4	7.1
	Angle of Internal Friction	(Degree)	35°-00'	36°-00'	28°-40'	26°-00'
Consolidation Amount(△)	Moisture Content	(%)	12.8	13.2	18.2	19.0
	Dry Density	(t/m^3)	1.90	1.88	1.70	1.67
		(lb/cu ft)	119	118	106	104
Pore Pressure (P)	Load	△ (%)	3.0	4.0	4.0	5.0
	10 kg/cm^3	P (kg/cm^2)	1.0	1.5	2.2	3.0
	142 psi	psi	14.2	21.3	31.2	42.6
Permeability Coefficient		(cm/sec)	1.5×10^{-5}	1.5×10^{-5}	6.5×10^{-6}	2.0×10^{-7}

Table 2. Properties of Stockpiled Materials.

	Maximum Size	(mm)	150.0
Gradation	-4 #	(%)	40.0
	-40 #	(%)	12.5
	-200 #	(%)	7.3
	-0.005 mm	(%)	4.0
Specific Gravity			2.65
Natural Moisture Content		(%)	10.0 - 14.0
Standard Compaction	Optimum Moisture Content	(%)	14.0
	Max. Dry Density	(t/m^3)	1.87 (117 lb/c.f)
Shearing Strength	Cohesion	(kg/cm^2)	0.55 (7.8 psi)
	Angle of Internal Friction	(Degree)	34° - 10'
Consolidation (Load $10 kg/cm^2$)	Consolidation	(%)	4.0
	Pore Pressure	(kg/cm^2)	1.6 (22.8 psi)
Permeability Coefficient		(cm/sec)	$<1 \times 10^{-5}$

Twenty ton sheepfoot rollers are being used for core zone compaction. Of the 2,000,000 m^3 (2,620,000 cu. yd.) of all materials placed in July-December, 1958, 250,000 m^3 (326,000 cu. yd.) was core material. Of the 203 tests made for moisture content of core materials placed during this period, 80.2% of the total showed that compaction was made on the dry side of the optimum.

Mr. W. E. Collins acts as a consulting engineer in the design of the dam, while engineers of Guy F. Atkinson Company are giving technological assistance relative to its actual construction work.

ROCKFILL DAMS: KAJAKAI CENTRAL CORE DAM AFGHANISTAN[a]

Discussion by F. L. Lawton

F. L. LAWTON,[1] M. ASCE.—Kajakai Dam is a classic example of one of the outstanding characteristics of the rockfill dam: its applicability to locations remote from sources of supply for equipment, cement and lumber but with ample rock and sufficient volumes of soil suitable for an impervious core. Kajakai is also typical of another characteristic: broad flexibility of design permitting most economic combination of available indigenous materials.

The overall cost of the dam, including all structures and appurtenant work constructed in the first stage, is given as about three dollars per cubic yard for a total quantity of earth and rock in the main embankment of 4,200,000 cubic yards. This would seem to be an extremely low cost considering the relative isolation of the site, but is no doubt explained by the composition of the work force, with only 69 foreign personnel, presumably all supervisory staff, and 1850 native workmen. It would add to the value of the paper if the author would indicate the relative productivity of the Afghan labour as compared with, say, average American labour of the same categories, and the relative rates of remuneration.

a. Proc. Paper 1735, August, 1958, by Glenn F. Sudmann.
1. Chf. Engr., Power Dept., Aluminium Labs. Ltd., Montreal, Canada.

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ROCKFILL DAMS: PERFORMANCE OF TVA CENTRAL CORE DAMS[a]

Discussion by F. L. Lawton

F. L. LAWTON,[1] M. ASCE.—The authors' treatment of the design and construction features of the Nottely, Watauga and South Holston dams is valuable. It is greatly enhanced by the discussion of operating experience.

The advantages of the central-core rockfill dams are well stated. The authors might have added that these probably apply equally well to rockfill dams with sloping impervious core. The type is essentially dictated by the relative availability of suitable material for the core. Both types appear much more resistant, inherently, to earthquake damage than any type of concrete dam.

Reference is made to the types of rock used for fill at the three dams, i.e. quartzite with 20 per cent mica schist at Nottely, quartzite at Watauga, and sandstone with some shale at South Holston. It would add to the value of the paper if the authors would give a brief account of the principal geological features in the cross sections at the sites. Reference to strike and dip of the formations would be useful.

The unequal settlement of the central core and rock shoulders, which has been noted at the three dams, particularly at Watauga and Nottely, gives rise to the question: Do the rock shoulders in reality slide, to a degree, along the contact zones? Would the authors care to comment on this point? This phenomenon appears to be a weakness of the central-core rockfill dam as compared with the sloping-core type.

The data reported for the lateral deflection is interesting. The rebound at Nottely dam and the upstream deflection for South Holston dam are noteworthy and confirm observations elsewhere. The implication, for South Holston, "The upstream deflection might have been due to tilting of the dam foundation during reservoir loading" is an interesting one and might be the explanation of what is, otherwise, difficult to explain.

Could the authors explain the significance of the broken-line portion of the deflection curve in Fig. 4?

a. Proc. Paper 1736, August, 1958, by George K. Leonard and Oliver H. Raine.

1. Chf. Engr., Power Dept., Aluminium Labs. Ltd., Montreal, Canada.

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ROCKFILL DAMS: SALT SPRINGS AND LOWER BEAR RIVER CONCRETE FACE DAMS[a]

Discussions by Carlos Tercero, Geoffrey Davey, Masatoshi Kawase, Tatsuo Mizukoshi and F. L. Lawton

CARLOS TERCERO.[1]—Salt Springs and Lower Bear River Dams provide invaluable experience for the designers of rockfill dams, and the knowledge of the designs and the construction methods employed as well as the knowledge of the settlements and leakages recorded, as presented by Mr. I. C. Steele and Mr. J. B. Cooke, will contribute to the improvement of future dams of this type.

In Mexico, the Federal Electricity Commission built, between 1954 and 1956, the 180-ft high Pinzanes rock fill dam, for its Tingambato hydroelectric project, its main object being to provide a reservoir for the hourly regulation of the flow used for the three Francis type turbines of 72,000 HP each under an average head of 1,620 ft. Due to this high head on the power plant the water has also a high value and minimum leakage is desired.

The Pinzanes dam site is located in an active volcanic zone composed of granite type rock of greenish color and not very sound, except in a few places that were used as a quarry from which rock was blasted at three or four different levels in order to choose the best kind of rock to be sent by trucks to the dam site, and dumped from a minimum height of 50 feet. The rock sent from the quarry was not larger than 2 tons and the average size was around 0.25 tons. Much of the rock was broken by impact in the 50 ft dumping from the trucks, and the fines as well as the smaller pieces of rock were carried into the voids of the larger pieces by the water used liberally and at high velocity through nozzles fed by electric pumps. The water jets were directed to the falling rocks and later to the pieces already in place, in order to keep their surfaces free from dust or from chips and to provide a suitable contact for the next loads of rocks. The water was used at an average of 7 to 10 cubic feet of water for one cubic foot of rock.

Fig. 1 shows the plan, developed view of the face, and section of the dam. In plan the dam is curved with an 870 foot radius to the centerline of the 16.5 foot wide crest. The upstream slope is 1.2:1 and downstream slope 1.3:1. The thin concrete face is on a layer of hand placed rock of uniform 6.5 foot thickness. The slabs are reinforced with a double mesh of one-inch round steel at 14 inches centers in the lower slabs and one layer at same spacing in the slabs near the top of the dam. The joints between two slabs are of the flexible and impervious type used in the Salt Springs Dam, made with a copper sheet embedded on each slab and U shaped in the center, keeping a slot of one

a. Proc. Paper 1737, August, 1958, by I. C. Steele and J. B. Cooke.
1. Engr. of Comision Federal de Electricidad, Mexico.

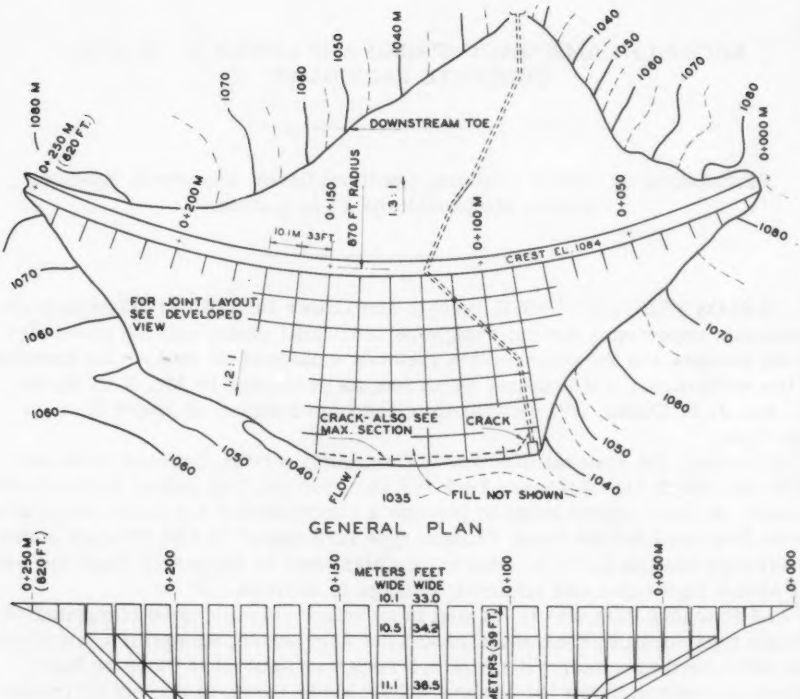


FIG. 1 PLAN VIEW AND SECTION - PINZANES DAM

inch between the slabs, that finally was filled with asphalted celotex and with asphaltic compound in the upper two inches of the slot. On the slabs located near the face of the hill additional flexible joints were provided parallel to the cutoff joint.

Along the cutoff, on the sandy bed of the river, it was necessary to excavate more than was originally planned, and the original sand and gravel was replaced with masonry more resistant to settlements than the surrounding rockfill; as there was no flexible joint on the slabs at the top of this masonry; the result was a crack on the slab soon after the concrete was poured. This crack was treated very easily with asphaltic compound and finally covered with a clay fill, Fig. 1. Another crack in the concrete was located in one slab in which the reinforcing steel was wrongly and rigidly connected with the rigid wall of the unwatering conduit located at the bottom of the dam. There is no doubt that the leakage through the face of the Pinzanes Dam is through these two cracks, that is, at the two places where the slabs were not flexible enough to follow easily the settlement of the rock fill.

When the reservoir was first filled, water soon appeared at the foot of the downstream face with the same brown color of the impounded water. Two or three days later, a second spring was located near the first, but with a colorless appearance. It was very clear that the first water came from the cracks of the concrete slabs, and that the second came through the hill. The maximum initial leakage amounted to 12.5 cfs but gradually it was reduced down to 5.5 cfs from which almost 50% comes through the foundation. The reduction was obtained by pouring small amounts of coarse sand followed by fine sand, through a 6-inch diameter pipe laid on the upstream face of the dam, from the crest to the site of the cracks.

The capacity of the Pinzanes reservoir is only 4,300 acre feet so that it can readily be unwatered, and it was easy to repair the cracks when the power plant was not yet in service; but now it is not desirable to empty the reservoir down to the very bottom to make the repairs, and although the water is valuable in Pinzanes as explained before, it does not pay to stop the plant for about two weeks for the saving of 2 or 3 cfs. Nevertheless, a method used by the Pacific Gas and Electric Company to detect the position of the cracks and to fill them without emptying the reservoir will be tried in the future.

The main object of this discussion is to point out the benefits derived from the experience obtained in the Salt Springs Dam in the design and construction of the economic and very satisfactory Pinzanes Dam. The design and supervision was made by Mr. Jesus Chavez Solano, from the Comision Federal de Electricidad of Mexico, after a careful study of the Salt Springs features and experience, especially those regarding the liberal use of water during the dumping of the rock, and in the provision of flexible joints near and paralleling the abutments.

The crest of the dam was built to a crest profile based on an overbuild of 1.5 per cent of the height of the point above the profile of the cutoff wall. However, such settlement is far from being reached, and this dam can be ranked among the most successful, so far as we know, because even after the earthquake of July 28, 1957, the dam did not show damage or cracks. The intensity of the earthquake was of the fifth degree—Mercalli System, with an acceleration between 25 and 50 millimeters per second per second.

GEOFFREY DAVEY,[1] M. ASCE.—The details in this paper of the settlement behaviour of the Salt Springs Dam, and the comparison of this behaviour with that of the Lower Bear River Dams, provide matter of great interest to engineers in Australia where, in many cases, rockfill is the only economical form of construction available. It is surprising to see the amount of settlement which did occur in the Salt Springs Dam during and immediately after construction. It might be asked whether such settlement would have occurred had the construction been carried out in small lifts of the order of 50' instead of the high lifts which were used. It should be possible with lower lifts to obtain full settlement in each layer during placing and hence to avoid later settlement.

The use of a reinforced concrete face on a placed stone up to 15' thick has obviously given very good performance despite the movement which has occurred in the fill, but it would be interesting to know the amount of leakage which has taken place through the dam. It would appear that in a controlled river system with numerous dams on one stream, that substantial leakage would not be a serious item, whereas such leakage would be quite serious for a water storage dam in an arid area of low rainfall.

MASATOSHI KAWASE.[2]—The author's paper which presents concise information on concrete face rockfill dams is very valuable to the engineers interested in such type of dams. The writer has prepared a brief discussion on the basic paper, presenting some data on the concrete face rockfill Ishibuchi Dam in a manner that it may be conveniently compared with that presented by the authors on Salt Springs and Lower Bear River Dams. It is hoped that the discussion may add something to the general pool of information on the subject of rockfill dams.

Ishibuchi Dam of the concrete face rockfill type, located on a tributary of the Kitakami River, was completed in 1953 by the Ministry of Construction. The main reasons for adopting this type at this site were unfavorable geological conditions and difficulty of procuring cement at that time. Fortunately, a sufficient supply of rock was available from a quarry consisting of dacite, while sufficient core materials were not obtainable nearby. Therefore, the concrete face rockfill type was adopted. Physical data are shown in Table 1.

Design Features

The mean slope of the downstream face, 1.6:1 was determined in consideration of stability against earthquakes. From the economic point-of-view, it would have been better to adopt a more gentle slope on account of the finishability of the face. The slope of the upstream face which is concave towards the reservoir varies from 1.4:1 to 1.2:1 and the axis of the dam in the plan is slightly curved convexly towards the reservoir. To effectively reinforce the parts where the differential settlement of the dam is greatest, the thickness of the placed rock, 3 m (9.8 ft.) at the crest, is increased linearly to 5 m (16.4 ft.) at the portion 20 m from the crest and it is made 5 m from this portion down to the bottom. The facing slab is of reinforced concrete and its thickness varies from 40 cm (15.8 in.) at the crest to 60 cm (23.6 in.) at

1. Cons. Engr., Gutteridge, Haskins & Davey, Sydney, Australia.
2. Chf., River Div., Tohoku Regional Const. Bureau, Ministry of Con., Tokyo, Japan.

Table 1. Physical Data of Ishibuchi Dam

Items	Data
Year completed	1953
Effective storage m^3	11,960,000 (9,800 acre feet)
Crest elevation m	323 (1060 ft.)
Height at axis m	53 (174 ft.)
Crest length m	345 (1130 ft.)
Surface area of face m^2	11,490
Slopes (horiz. to vert.)	
Upstream	1.2 to 1.4
Downstream	1.6 (mean)
Placed rock thickness m	
Top	3 (9.8 ft.)
Bottom	5 (16.4 ft.)
Quantities m^3	
Excavation	overburden 131,500 (172,000 cu.yd.) rock 36,000
Dumped rock	361,400 (473,000 cu.yd.)
Placed rock	49,900 (65,300 cu.yd.)
Concrete (slab & keyway)	8,800 (11,500 cu.yd.)

the base. The amount of reinforcement was about 0.5% of the slab section and two layers were adopted in consideration of resistance to temperature changes and settlement of the rockfill dam. The facing slab was generally divided into 10 meter (32.8 ft.) squares by the horizontal and vertical joints, and exceptionally 3 to 5 m (9.8 to 15.4 ft.) horizontal joints were adopted in the part adjacent to the river bed where more flexibility was needed. The idea above mentioned is similar to that of the hinge slab adopted in Lower Bear River Dam. The plan, upstream view, cross section and details are shown in Fig. 1.

Foundation Treatment

Geologically the dam site consisted mainly of liparite decomposed or jointed considerably, and the foundation rock was carefully grouted. A total of 1,587 tons of cement was injected through 9,640 meters of bored holes for curtain grouting. This shows the unfavourable geological condition of the site in comparison with that of the sites of both Salt Springs and Lower Bear River Dams.

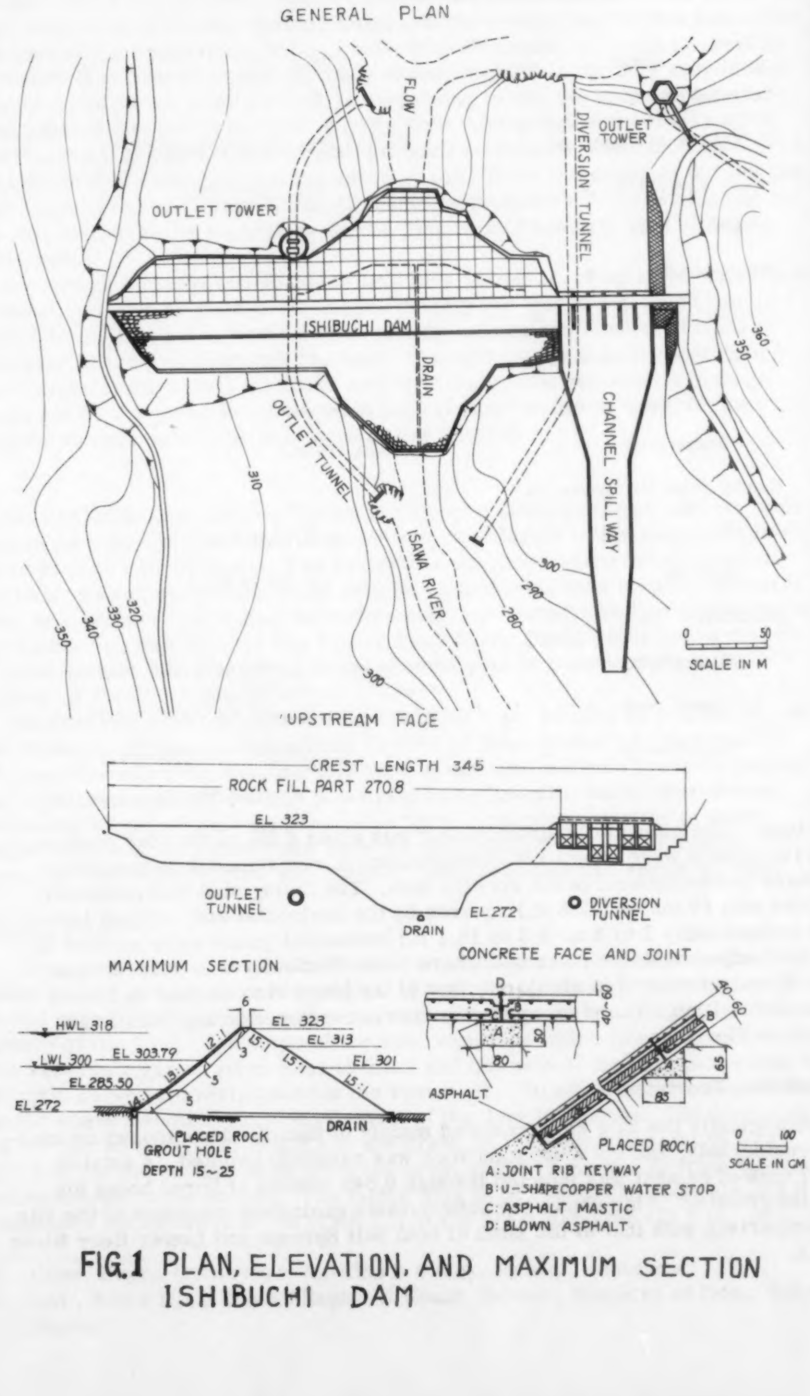


FIG. 1 PLAN, ELEVATION, AND MAXIMUM SECTION
ISHIBUCHI DAM

Quarrying, Dumping and Sluicing

Rocks were quarried by coyote blasting method at a quarry 1.5 kilometers upstream of the dam site and transported to the dam site on the side-dump cars which were pulled by diesel locomotives on two railroads constructed at El. 299 meters and El. 323 meters. The properties of rocks were as follows: specific gravity 2.5, absorption 4.1% and compressive strength 1,290 kg/cm^2 (18,000 psi) to 1,610 kg/cm^2 (23,000 psi). Fill-rock was dumped mainly from a bridge spanned across the river, which was constructed at the 299-meter level in the first stage and raised up to the 323-meter level in the secondary or final stage.

Uniformity of grading of fill-rock along the axis of the dam was obtained by this device of dumping. In order to expedite the settlement of rockfill in the upstream part, fill-rock was dumped on this part prior to the dumping on the downstream part. Maximum dumping height was 29 m (95 ft.) and such fines as reported on the paper of Salt Springs Dam did not accumulate on the top zone of the lift. Sluicing water was jetted in the quantity of not less than two times the volume of dumped rock and at the pressure of 7 kg/cm^2 (100 psi). However, effectiveness of sluicing by such volume of water was not sufficient. Dumping was started in May 1950 and finished in June 1953. Placing of rocks in the rubble wall section was started in July 1951 and came to an end in March 1953.

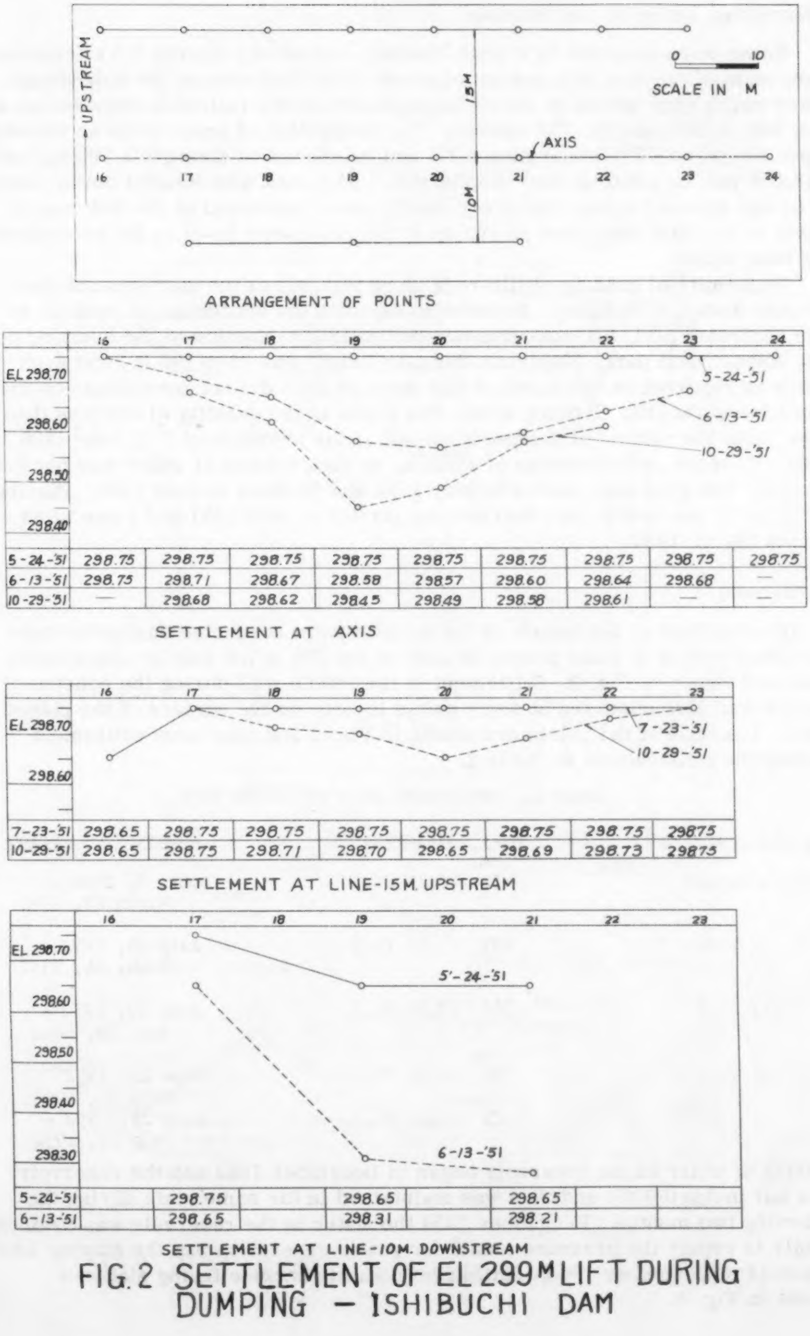
Settlement

Observations of settlement of the rockfill were conducted during the construction period at some points located on the 299 m lift and the observation data are shown in Fig. 2. Settlement of the rubble wall during the construction period was also observed at some points located on the surface of the placed rock. Location of the points are shown in Fig. 3 and maximum settlement during the construction in Table 2.

Table 2. Settlement Data of Placed Rock

Location of Points	Max. Settlement (mm)	Observation Period
Near crest-18	58 (0.19 ft.)	Oct. 6, 1952 - March 23, 1953
" II - 20	137 (0.45 ft.)	July 28, 1952 - Sept. 24, 1952
" III - 18	203 (0.67 ft.)	June 22, 1952 - Aug. 18, 1952
" IV - 74	74 (0.24 ft.)	June 22, 1952 July 21, 1952
" V - 23	23 (0.08 ft.)	June 22, 1952 - July 14, 1952

Storing of water in the reservoir began in December 1953 and the reservoir was full in April 1954 and then was maintained in the same state during the following two months. In October 1954 the water in the reservoir was drained wholly to repair the pressure tunnel for power generation and the storing was resumed in December 1954. Settlement data of concrete facing slab are shown in Fig. 4.



	16	17	18	19	20	21	22	23	24
5-24-'51	298.75	298.75	298.75	298.75	298.75	298.75	298.75	298.75	298.75
6-13-'51	298.75	298.71	298.67	298.58	298.57	298.60	298.64	298.68	—
10-29-'51	—	298.68	298.62	298.45	298.49	298.58	298.61	—	—

SETTLEMENT AT AXIS

	16	17	18	19	20	21	22	23
7-23-'51	298.65	298.75	298.75	298.75	298.75	298.75	298.75	298.75
10-29-'51	298.65	298.75	298.71	298.70	298.65	298.69	298.73	298.73

SETTLEMENT AT LINE-15M. UPSTREAM

	16	17	18	19	20	21	22	23
5-24-'51		298.75		298.65		298.65		
6-13-'51		298.65		298.31		298.21		

SETTLEMENT AT LINE-10M. DOWNSTREAM

FIG. 2 SETTLEMENT OF EL 299 M LIFT DURING DUMPING — ISHIBUCHI DAM

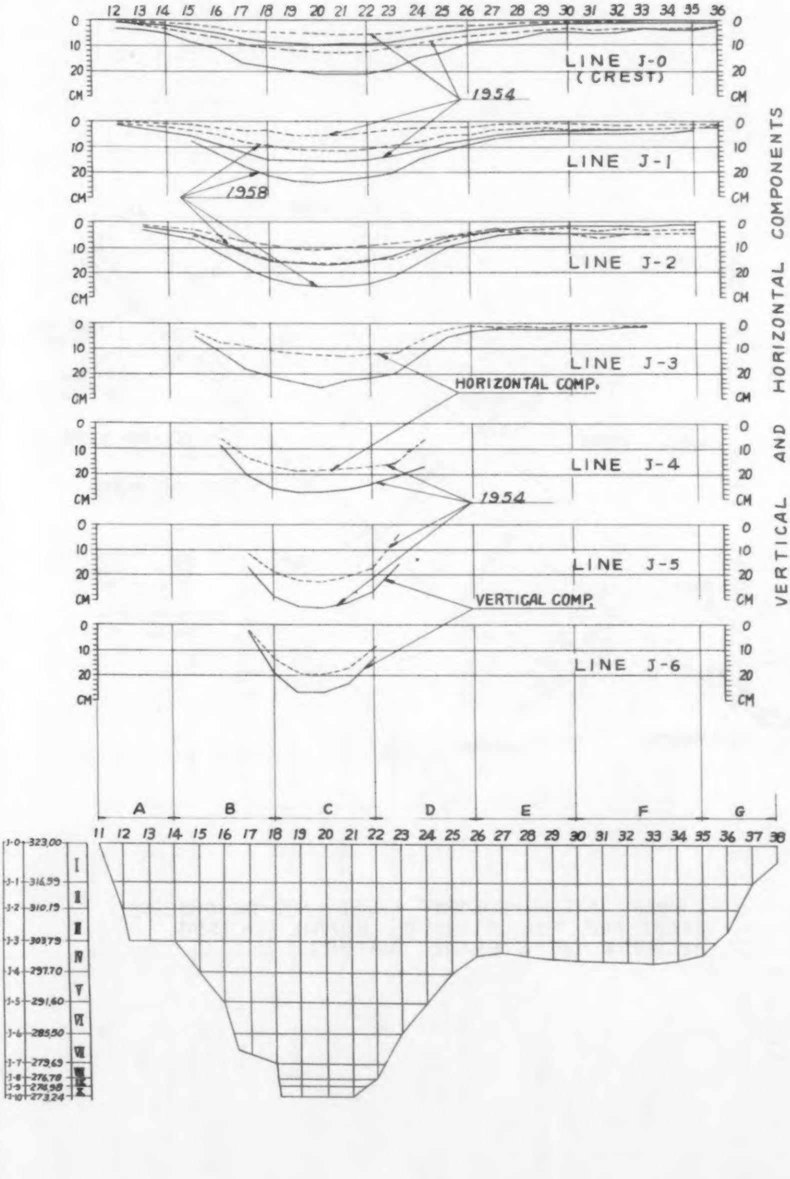


FIG. 3 VERTICAL AND HORIZONTAL MOVEMENT OF CONCRETE FACE – ISHIBUCHI DAM

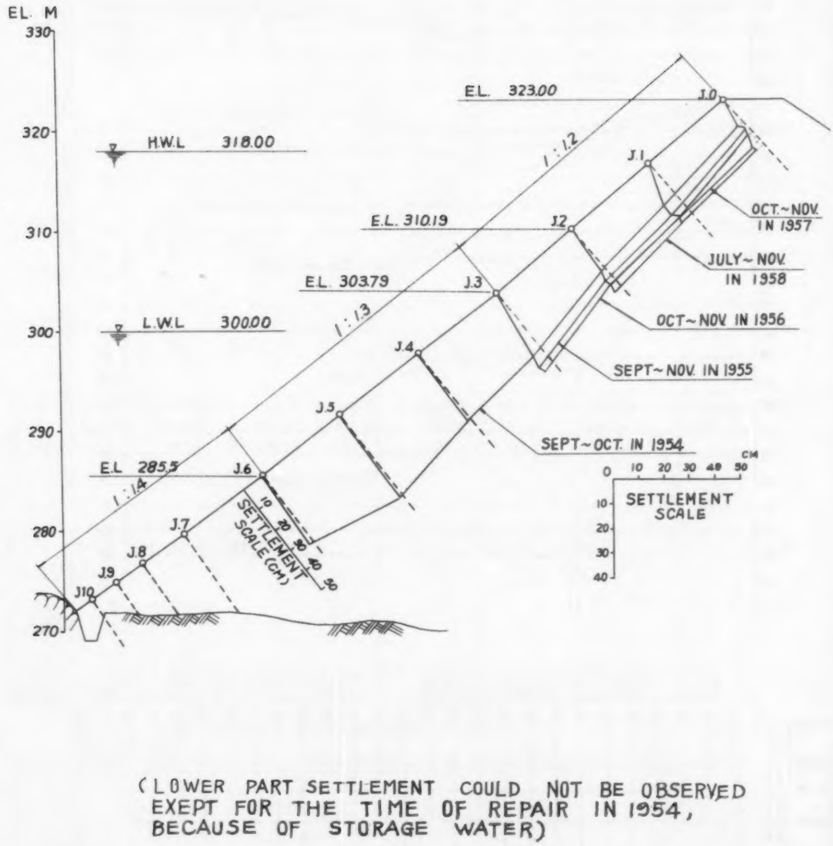


FIG. 4 SETTLEMENT AT MAXIMUM SECTION
STATION NO.20 — ISHIBUCHI DAM

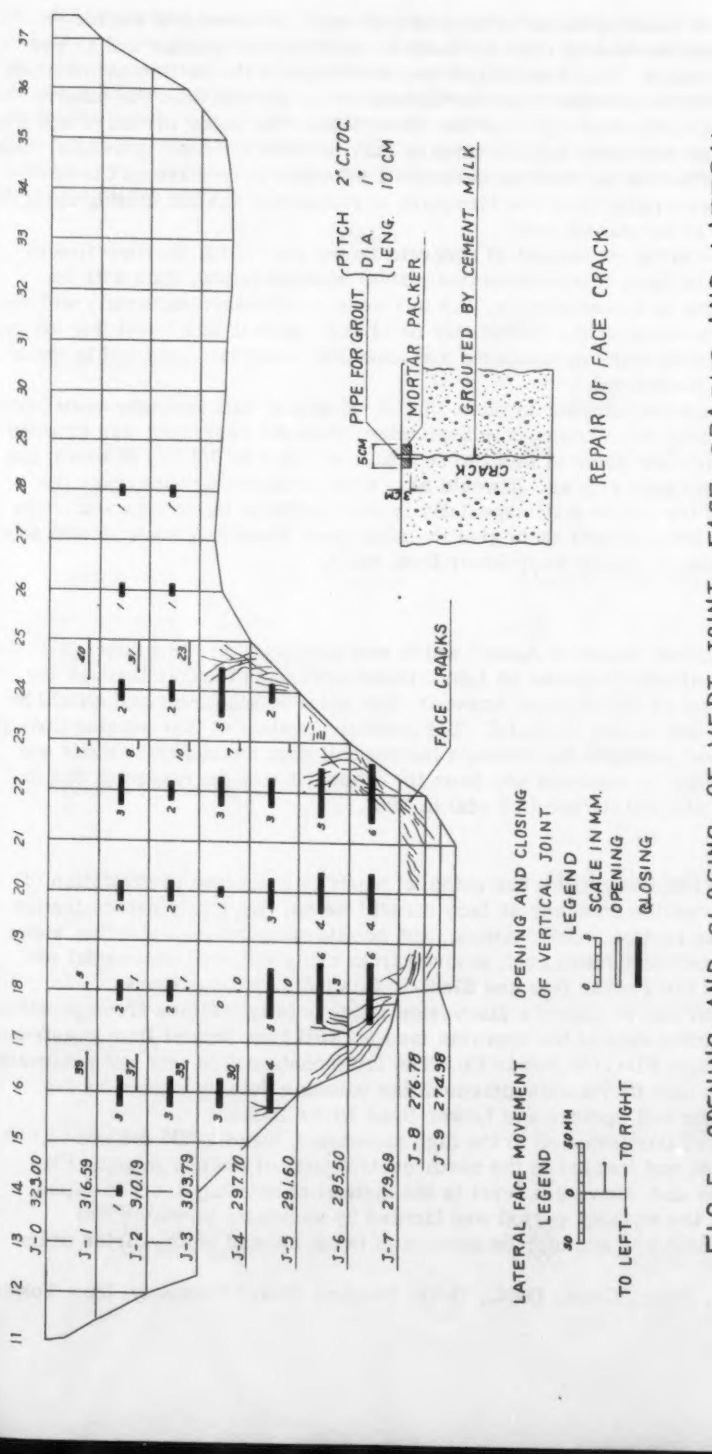


FIG.5 OPENING AND CLOSING OF VERT JOINT FACE CRACKS AND LATERAL MOVEMENT - ISHIBUCHI DAM

In point of relationship of settlements between the crest and the lower parts, the data show a similar tendency to those of Salt Springs and Lower Bear River Dams. It is considered that this shape of the settlement curve is due to the following reason: the settlement of the portion near the base occurs more quickly than that near the crest, where the water pressure and own weight of dam are considerably smaller than those of the other portions. Then it is considered the form of the concrete face which is concave on the upstream is favourable from the viewpoint of resistance against sliding along the surface of the placed rock.

In regard to the movement of concrete facing slab along the direction of the axis of the dam, it was found that all the vertical joints, each with an initial opening of 2 centimeters, (0.8 in.) were compressed uniformly and contracted a maximum of 0.6 centimeter (0.23 in.). Also it was found that all the horizontal joints were compressed and asphaltic materials inserted in these joints were pushed out.

Several cracks, similar to those that developed at Salt Springs, were found in the slab near the abutments in September when the reservoir was emptied (Fig. 5). Although some of them were 3 to 5 mm (0.1 to 0.2 in.) in width, the majority were hair cracks. It would have been preferable to increase the thickness of the rubble wall especially in such parts as those adjacent to the steep abutments, as well as to give the slab more flexibility such as was successfully done at Lower Bear River Dam No. 1.

Leakage

The pamphlet "Dams in Japan" which was presented to the engineers at the Sixth International Congress on Large Dams describes leakage through the Ishibuchi Dam as 226 lit/min; however, this value is incorrect and should be replaced by 226 lit/sec (8.0 cfs). The leakage consists of that passing through the foundation rock and that through the rockfill dam embankment itself and it is impossible to separate one from the other. It was decreased to 204 lit/sec in 1957 and 190 lit/sec (6.7 cfs) in 1958.

TATSUO MIZUKOSHI.[1]—The authors' paper is a concise presentation of factual information on concrete face rockfill dams, especially determination of dam cross section, rock dumping, and relationship between sluicing water to dumped rock and settlement, derived from many years of successful experiences of the Pacific Gas and Electric Company by the authors.

The writer has prepared a discussion which briefly outlines some practice and construction data of the concrete face rockfill type Nozori Dam constructed by the Tokyo Electric Power Co. The facts contained herein are presented with the idea that they may supplement the valuable data presented by the authors on the Salt Springs and Lower Bear River Dams.

The Nozori Dam situated in the high mountains, about 1,500 meters (4,910 ft.) in altitude and located in the north-central part of Honshu Island. Elevation of this dam above sea level is the highest of existing dams in Japan. At this site, the working period was limited by snowfalls in winter and frequent rainfalls in summer on account of being located on the divide of the

1. Chf. Civ. Engr., Const. Dept., Tokyo Electric Power Company, Inc., Tokyo, Japan.

Pacific Ocean side and the Japan Sea side, and the transportation of cement and other materials was minimized from the topographical remoteness. A rockfill type was adopted from the economic viewpoint and in consideration of possible future heightening to increase the storage capacity in relation to a pumping-up scheme. And a concrete face rockfill type was selected chiefly because of the above-mentioned unfavorable climatic condition as well as unavailability of suitable impervious fill materials near the dam site. Physical data are shown in Table 1.

Table 1

Physical data of the Nozori Dam

Item	Data
Year completed	1956
Catchment area	16.6 km^2 (6.4 sq. mi.)
Effective storage	28,400,000 m^3 (23,000 acre feet)
Crest elevation	1,517 m (4,980 ft.)
Height at axis	44 " (144 ft.)
Crest Length	152.5 " (500 ft.)
Surface area of face	4,280 m^2 (46,000 sq. ft.)
Slopes (horiz. to 1 vert.)	
Upstream	1.3
Downstream	1.5 (mean)
Placed rock thickness	
Top	3.0 m (9.8 f t.)
Bottom	3.5 " (11.5 ft.)
Facing slab thickness	
Top	0.3 m (11.8 in.)
Bottom	0.6 " (23.6 in.)
Quantities	
Excavation	46,400 m^3 (60,600 cu. yd.)
Dumped rock	160,200 " (210,000 cu. yd.)
Placed rock	17,200 " (22,500 cu. yd.)
Concrete	
Facing slab	2,400 m^3 (3,140 cu. yd.)
Cut-off wall	3,900 " (5,100 cu. yd.)

Design Features

The natural slope of the dumped rock was presumed to be 1.3:1 (horizontal to 1 vertical). The 1.3:1 upstream and 1.5:1 downstream face slopes were determined in consideration of the influence of earthquakes, and for the downstream face, the 1.3:1 face slope with two berms of 4.5 meters (14.8 ft.) each in width, was actually adopted from the natural slope of the dumped rock. The thickness of the crane-placed rock, 3 meters (9.8 ft.) normal to the face for the full section, was thought to be enough, in the case of a dam of this scale, to minimize the influence of water pressure and settlement, but in the lower part the thickness varied from 3.0 to 3.5 meters. The downstream face below the lower berm was paved with the thin placed rock. A plan and a cross section of the dam are shown on Fig. 1a and Fig. 1b. Actually, an overfill amounting to 2% of the dam height was adopted in consideration of the settlement of the fill-rock.

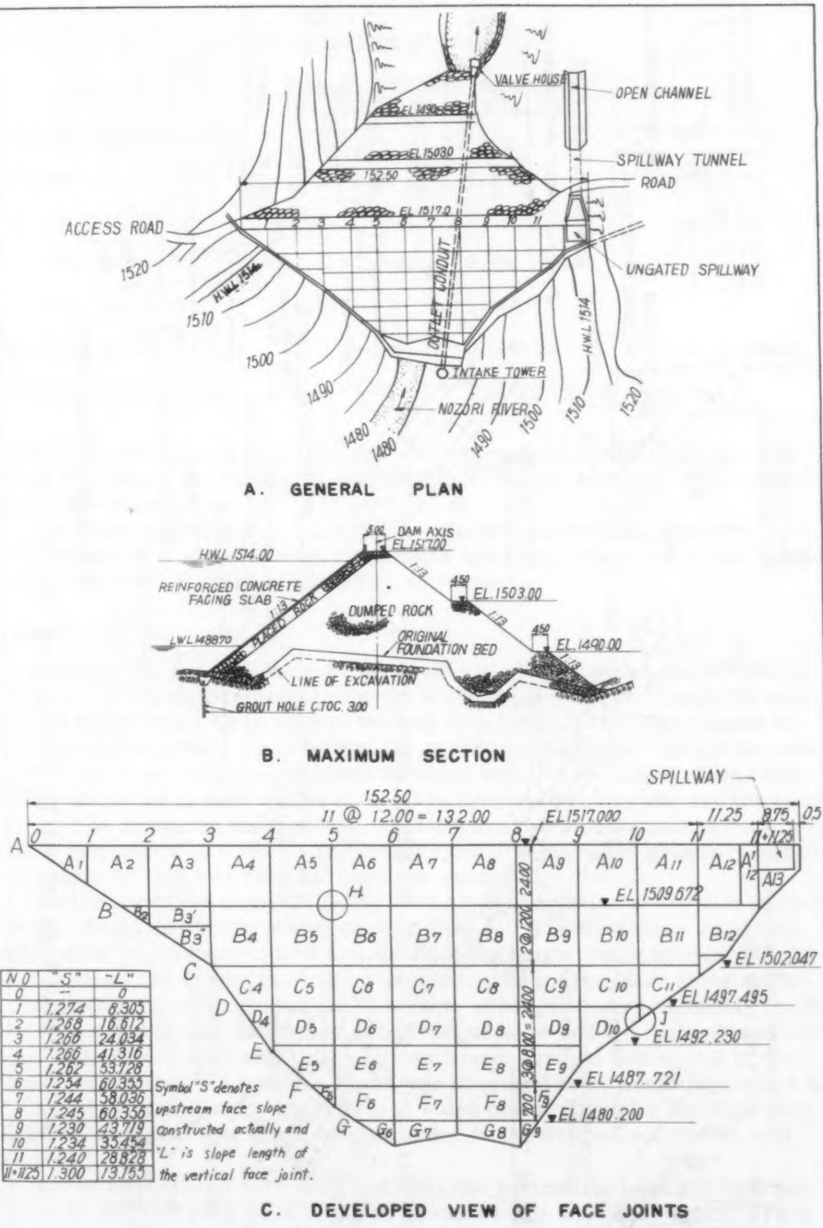
The face slab is of reinforced concrete. The slab thickness at the bottom was based on 1.5% of the water head. The amount of reinforcing bar was about 0.5% of the slab section and a single layer arrangement in the upper half and double arrangement in the lower half were adopted. The joints of the slab were arranged as shown on Fig. 1c. These were divided to 12 meters (39.4 ft.) square, 12 x 8 meters and 12 x 7 meters, excepting the perimetral part adjacent to the exposed foundation rock. Details of these joints are shown on Fig. 2. Vertical joints with U-shaped copper waterstops, filled with asphalt and packed at the surface with blown asphalt containing 1.5% of asbestine fiber, are designed 25 millimeters (1 in.) open to prevent crushing of joints. Horizontal joints with flat copper plates, filled with asphalt, are also designed 10 millimeters (0.4 in.) open. These flat waterstops were considered to simplify the cross connection of horizontal and vertical joints. Joint rib keyways are coated with asphalt milk before slab concrete is placed. The U-shaped copper waterstops are used at the perimetral joints of cut-off wall and facing slab, both faces of which are also separated by asphalt fillers.

Quarrying, Dumping and Sluicing

Rocks at the dam site and nearby are andesites and propylites. Quarried rocks, supplied from several quarries close to the left abutment of the dam, were obtained by the coyote hole method in 30 blasts with coyote holes totaling 1,120 meters in length. About 26 tons of Carlit (powder explosive of ammonium perchlorade) were used for blasting 215,000 cubic meters of rocks and the average explosive factor was found to be 0.27. Quarried rocks weighing 0.2 to 2 tons were used for the dumped rock. The physical properties of the rock are shown in Table 2.

The rockfill was dumped by rear-dump and side-dump cars in three stages of dumping heights, 13 to 17 meters (43 to 56 ft.). These dumping lines are at El. 1490, 1505 and 1517 meters, respectively. Rockfill was constructed in equal amounts in the two 6 month construction seasons of June - November 1954 and 1955. The maximum rate of dumping was 1500 cubic meters (1960 cu. yd.) per day.

Sluicing was specified to be at not less than three times the volume of rock being dumped. Sluice water was pumped up by two 700-hp pumps from the Nozori River, supplied through two 150-millimeter dia. steel pipes to the dumping sites and poured to the dumped rock by 64-millimeter dia. rubber hoses. Every rock was washed thoroughly with this sluice water concentrated



NO	"S"	"L"
0	—	0
1	1.274	8.305
2	1.268	16.612
3	1.266	24.034
4	1.266	41.316
5	1.262	53.728
6	1.254	60.355
7	1.244	58.036
8	1.245	60.356
9	1.230	43.719
10	1.234	35.454
11	1.240	28.828
11+11.25	1.300	13.155

FIG 1.—PLAN, SECTION AND DEVELOPED VIEW SHOWING JOINTS OF CONCRETE FACING SLAB

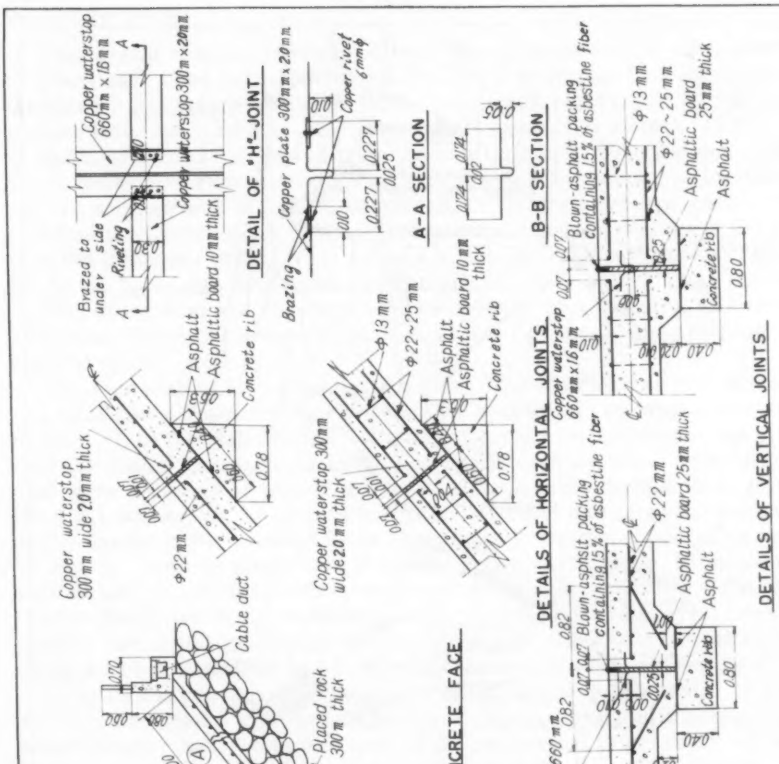


FIG. 2 – DETAILS OF CONCRETE FACING SLAB JOINTS

Table 2.
Physical Properties of Rock

Item	Date of test pieces sampled Jan., 1953	Aug., 1955	Remarks
Specific gravity	2,705	2,700	
Absorption	0.311%	0.500%	
Compressive strength			
Wet condition	3,515 kg/cm^2	2,590 kg/cm^2	
Dry condition	3,189 "	2,446 "	
Weathering test	-----	0.00023%	Tested by means of Na_2SO_4 solution
Modulus of elasticity	-----	757,000 kg/cm^2	Measured Statistically

from three points as it is placed and water consumed for this purpose was about four times the volume of dumped rock. The jet pressure of the sluice water was about 12 kg/cm^2 (170 psi.).

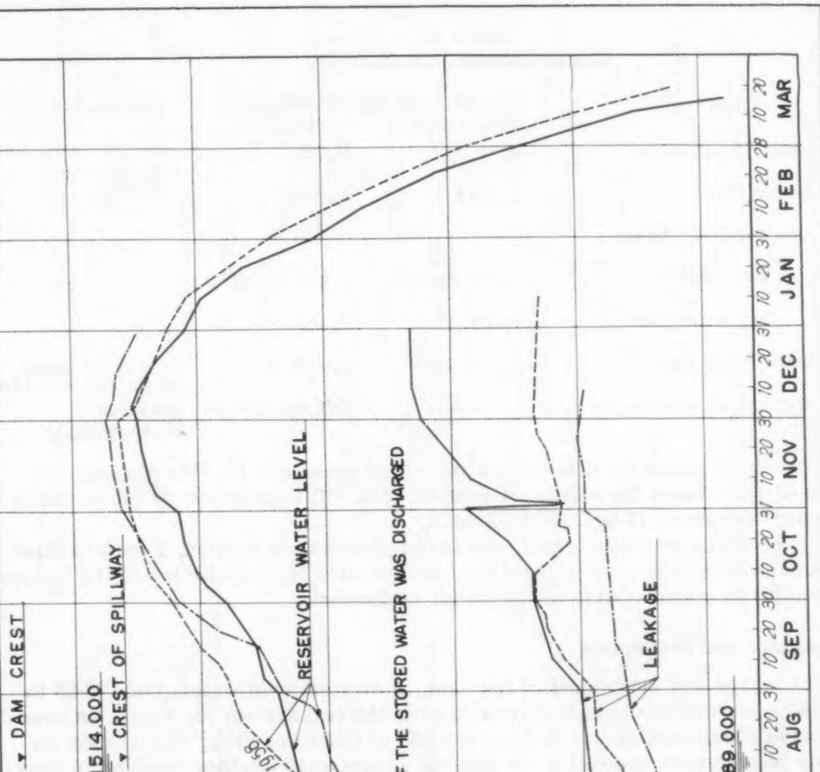
The rocks weighing 2 to 7 tons in the placed rock section, 3 meters thick, were placed by cranes at least two months after the completion of the dumped rockfill in consideration of the initial settlement.

Leakage and Settlement

Leakage and settlement of the dam, after storing of water, from 1956 to 1958 are shown on attached drawings. In the leakage curve, Fig. 3, an unexpected sharp decline was seen at the end of October, 1956. The reason for this phenomenon appears to be that the severe cold weather brought an unexpected heavy snowfall as the result of which some of the leakage was temporarily absorbed in snow on the streambed between the dam site and the leakage weir. The maximum leakage was recorded in 1956 which was less than 23 liters per second (0.8 cfs.). The leakage tends to decrease gradually and the maximum in 1958 was only 10 liters per second (0.4 cfs).

Settlement of the concrete facing slab after completion of the dam is shown on Fig. 4 and that of the placed rock on Fig. 5a. The first-year waterload settlement is greatest around four-tenth of the height which agrees with the Salt Springs and Lower Bear River measurements. The later aging settlement is thought to be greatest at the crest, although it was impossible to observe the facing slab movement on account of snow and ice that covered the slab in Aprils of 1957 and 1958. The maximum vertical settlement at the crest occurred at the joint No. 8 and was observed as 10 centimeters (0.4 ft.), 0.23% of the dam height, about 50% of which occurred during the first year. It is believed that this small amount is due to the dumped rock being well sluiced.

Three face cracks have occurred near the perimetral joint and spillway crest in the first year of service, as shown on Fib. 5b. The vertical crack developed from slab F5 to slab D5, the maximum width of which amounted to 1.5 millimeters (0.06 in.), was about 12 meters long. The horizontal crack parallel to the perimetral joint at the bottom of the dam was about 8 meters long and its maximum opening was observed as 0.5 millimeters (0.02 in.).


DAM CREST
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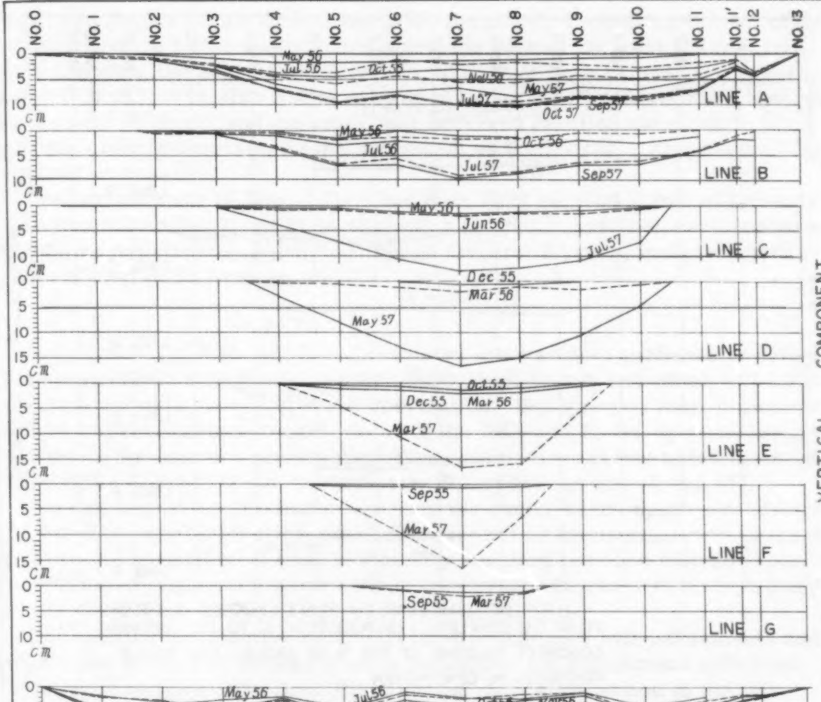
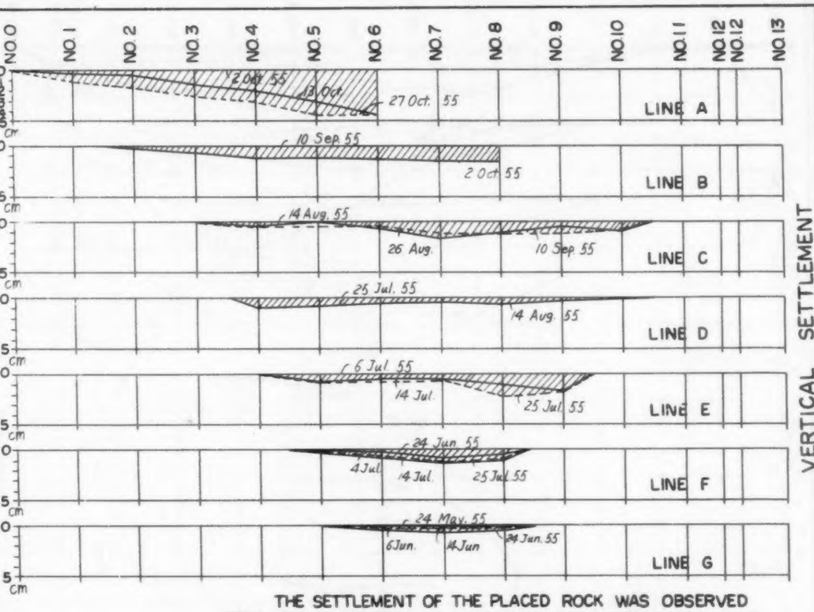


THE HORIZONTAL SECTIONS SHOW VERTICAL AND HORIZONTAL COMPONENTS. SETTLEMENT SCALE IS PREPARED IN CENTIMETER.

FIG. 4 — SETTLEMENT OF CONCRETE FACING SLAB



A. SETTLEMENT OF PLACED ROCK DURING CONSTRUCTION

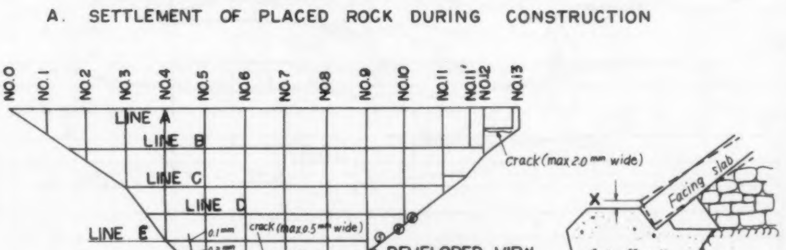


Table of edge rise "X" of the slab in centimeter

Date observed	Joint points observed ①	②	③	④	⑤	⑥	⑦	⑧
31 March. 1957	0	1.8	3.0	3.5	2.0	0	1.0	—
3 March. 1958	—	—	—	—	—	—	—	1.5

B. FACE CRACKS AND PERIMETRAL JOINT MOVEMENT

FIG. 5 – SETTLEMENT OF PLACED ROCK, FACE CRACKS AND PERIMETRAL JOINT MOVEMENT

These two cracks occurred because of the relatively unequal movement of the fill-rock and the latter was situated above the edge of the joint rib concrete below the slab, the edge of which was found to have been raised by 1.0 centimeter (0.4 in.). The slab A13 is supported by the cutoff concrete and concrete piers on the bedrock, and the crack that occurred parallel and close to the spillway crest is unrelated to the settlement of the rockfill. The cracks have required no repair.

The performance of Nozori Dam has been very similar to that of Lower Bear River Dam No. 1, which is of similar height. It has been very successful with no maintenance being required in the first 3 years, moderate settlement and negligible leakage.

F. L. LAWTON,[1] M. ASCE.—The authors have made a particularly noteworthy contribution to the Symposium on Rockfill Dams, one which will long remain a valuable source of basic information. Being drawn from experience with 14 impervious face rockfill dams in the P.G. and E. Co. system, their location in the massive granite formations of the Sierras has added materially to the value of the data due to similarity of foundations and of rockfill.

The opening of the vertical crack near the center of the upstream face of the 140-foot high Relief Dam, with its crest curved downstream, would appear to prove the soundness of what is virtually standard practice with all types of rockfill dams, especially those with an impervious face or a thin sloping impervious core, i.e. the upstream curvature of the axis.

The authors' correlation of construction procedures with settlements and deflections in the Salt Springs Dam is invaluable for comparison with later dams, such as the Lower Bear River Dams. It drew attention to the importance of effective sluicing with plenty of high-pressure water properly directed to eliminate accumulation of fines between contact surfaces and points of the rockfill.

The vertical and horizontal rebound of points, on the maximum section of the Salt Springs Dam, even though relatively small, confirms experience with other types of rockfill dams discussed in other Symposium papers. Apparently rockfill dams act as imperfectly elastic bodies, within narrow limits. Salt Springs Dam experience over 31 years, in respect of horizontal movement in the plane of the face (lateral settlement) of the dam, affords a useful basis for assessment of possible lateral settlement in rockfill dams with thin sloping impervious cores. It demonstrates the effect of steep abutments on lateral settlement.

The authors' analysis of the inter-relationship between the several components of movement in the mass of a rockfill dam is stated as:

> "As water pressure causes significant downstream and downward differential movement, a lateral readjustment of rock points must take place. It appears that this lateral readjustment of rock contacts, along with the active pressure within the rockfill and the high percentage of voids, causes the rocks to move toward the centre of the dam. In addition to these factors the basic vertical settlement should tend to develop a component toward the center due to the sloping abutments. - -

1. Chf. Engr., Power Dept., Aluminium Labs. Ltd., Montreal, Canada.

It seems that the high initial lateral movements are associated with the high normal settlement in the lower and central portion of the dam."

Drawn from experience with concrete face dams, the analysis seems to be particularly applicable to rockfill dams with thin sloping impervious cores. It emphasizes the necessity for upstream curvature of a rockfill dam and the trimming of steep slopes on the abutments, as well as the elimination of sharp transitions in abutment slopes.

The average maintenance cost for Salt Springs Dam at $8,000 per year is indeed moderate. Did the $2,000,000 estimated saving, in 1931 dollars, achieved in using a rockfill rather than a concrete dam, represent construction costs only?

ROCKFILL DAMS: DALLES CLOSURE DAM[a]

Discussion by F. L. Lawton

F. L. LAWTON,[1] M. ASCE.—The construction of the rockfill closure dam for The Dalles project represents a notable example of the adaptability of the rockfill dam to difficult sites. In this connection, it would be interesting if the author could indicate the probable increase in cost of the alternative closure.

It is clear from the paper that "the general plan developed for construction of the closure dam consisted of first constructing a dumped rock diversion fill to divert the river flow through eight skeleton units in the powerhouse, then completing the closure dam by raising the dumped rock fill and placing an impervious blanket on the upstream face". However, it is not immediately clear to what elevation the diversion fill was constructed, although the paper is interpreted to mean that the diversion fill was built up to a maximum elevation of 170 feet, approximately, above the foundation. Could the author clarify this point?

Reference is made to the fact the diversion fill withstood ". . . four annual flood flows before completion of the closure without appreciable erosion . . ." Could the author indicate the velocities which are known to have occurred over the surface of the diversion fill?

The use of the underwater television camera for examination of the diversion fill is another instance of the great utility of the camera, particularly when it can be used in such difficult conditions as the Big Eddy Pool. The procedure adopted for the placement of the blanket appears to be a very sound one. Could the author indicate the loss of material which is estimated to have occurred in placement? In connection with permeability of the sandy-gravely blanket, it is indicated ". . . the computed seepage loss of 100 to 170 cfs through the newly completed blanket was expected to decrease considerably with time as silt is deposited on the outer slope". Is there any indication of this proving to be the case?

The paper also makes reference to the use of sonic depth finders for the underwater surveys made prior to the start of construction, with the lead line being used after diversion. Could the author indicate the relative accuracy of the two methods as found under the site conditions?

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a. Proc. Paper 1738, August, 1958, by Robert J. Pope.

1. Chf. Engr., Power Dept., Aluminium Labs. Ltd., Montreal, Canada.

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ROCKFILL DAMS: REVIEW AND STATISTICS[a]

Discussion by F. L. Lawton

F. L. LAWTON,[1] M. ASCE.—The authors are to be congratulated on an excellent analysis of the basic features and types of, and statistics on, rockfill dams.

The paper notes that "Constructed as expedients during pioneer days in the West, several of those early, though smaller, rock-fill dams remain as functioning monuments to ingenuity and engineering enterprise." This is true. However, it should not be deduced therefrom that these early dams were born "out of the blue". It is entirely probable their line of descent is from the rock-fill timber-crib dams which the early comers to eastern North America built to assist in logging operations. Some of these reached the notable heights of 70 to 100 feet.

The authors have provided an excellent classification of rock-fill and earth-fill dams but it is questionable if the use of the terminology "deck type" is sound. The accepted meaning of the word "deck" is a platform or covering extending horizontally across a vessel. While there are variants in meaning, depending on the country, the concept as applied to rock-fill dams is wrong. It is suggested "membrane" is more correctly applicable to those rock-fill dams which the authors have classified as deck type.

It is particularly interesting to note the authors have suggested that ". . . elimination of an excess of fines remaining in the run-of-quarry output might be accomplished by the use of (dump) trucks with slotted bottoms, thus permitting fines to be shaken out of the truck loads en route from quarry to dumping site." Certainly the selective dumping of quarry-run rock is relatively ineffective and decidedly uneconomic as a means of eliminating deposition of fines, although ample properly-directed high-pressure sluicing can go a long way towards overcoming settlement trouble arising from excess fines.

a. Proc. Paper 1739, August, 1958, by John B. Snethlage, F. W. Scheidenhelm and Arthur N. Vanderlip.

1. Chf. Engr., Power Dept., Aluminium Labs. Ltd., Montreal, Canada.

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ROCKFILL DAMS: THE DERBENDI KHAN DAM[a]

Discussion by S. Sakurai

S. SAKURAI.[1]—The writer has read the author's paper with particular interest since he is engaged on the Makio Dam, of similar type and magnitude to Derbendi Khan Dam. The writer will summarize the site conditions and the solutions at Makio in order to contribute to the valuable information being made available through the ASCE Symposium.

Makio Dam is a multi-purpose project located on the Otaki River, near Nagoya, Japan. The dam is of thin central core and alluvial-rockfill shell type, of 350 foot height, as shown on Fig. 1, 2, and 3. Construction began in late 1957, embankment construction will begin in July of 1959, and completion is scheduled in December of 1960.

Geology

The formation consists of strata of sandstone, slate and chert of Paleozaic, and diabase inserted in some slate. The strike is in general up-downstream direction and dip 30° - 70°. There is a fault at the center of the river and several smaller faults of crushed material and clay, A, B and C of Fig. 3. There are four steps, each several meters thick, of terrace materials of andesite and granite sands and gravels. The river bed deposit is 25 meters thick with extensive boulders near the ground surface and more sands than gravel at the bottom. Nearly 100% CO_2 gas and a little H_2S gas is emitted at the site and ground waters contain free CO_2 in nearly saturated condition, Ca O, Mg O, Fe_2O_3, S_i O_2, and $A\ell_2$ O_3 as dissolved from the rocks. Gas and low pressure springs are general in the area.

A pervious saddle near the damsite, consisting of an ancient channel of alluvium, volcanic ash and pumice, was thoroughly investigated and accepted.

Construction Materials

Materials conveniently available for embankment construction are: rock from spillway excavation (sandstone); transition material of well graded sand-gravel-cobble-boulder alluvium; core material from downstream river terraces (5 - 15 m. thick) of volcanic ash and breccia of andesite, chert, slate, sandstone, etc., and filter material from selective borrowing in the above alluvium deposits.

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a. Proc. Paper 1741, August, 1958, by Calvin V. Davis.

1. Director of the Aichi Irrigation Public Corp., Nagoya, Japan.

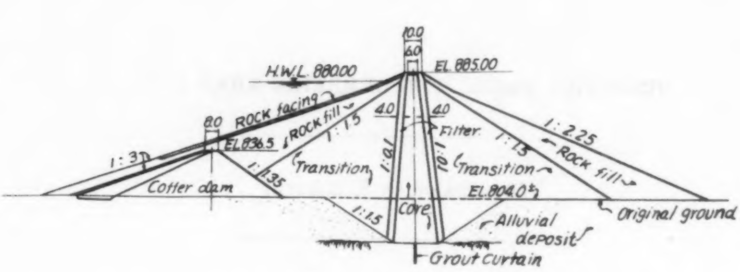

FIG 1. MAKIO DAM EMBANKMENT SECTION

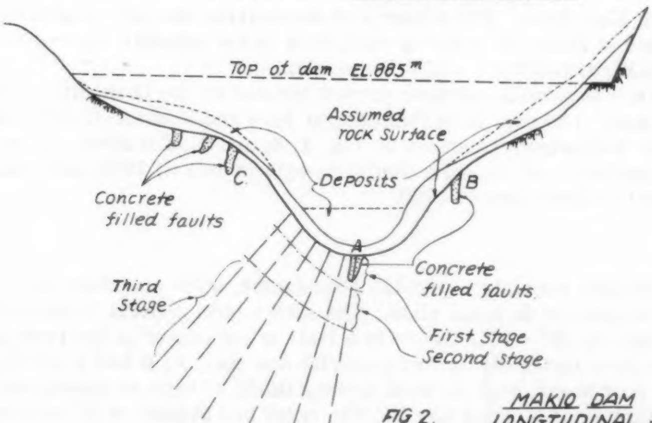

FIG 2. MAKIO DAM LONGTUDINAL SECTION

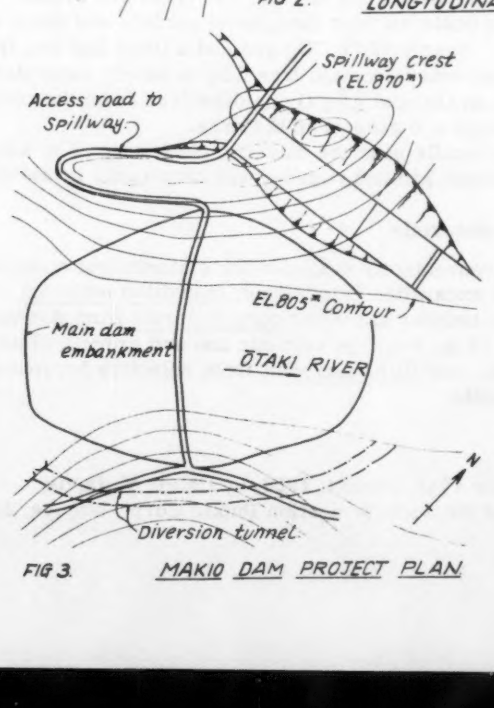

FIG 3. MAKIO DAM PROJECT PLAN

Core Material

The core material is unusual and of special interest. Grains of size less than 2 mm consist of volcanic ashes and a small amount of flakes of breccia and small pieces of pumice or talus. Classification is sandy loam and color yellowish brown. Specific gravity is 2.7 to 2.8 and grading is 5 - 15% Clay (0 - 0.005 mm), 10 - 34% silts (0.005 - 0.05 mm), 15 - 35% fine sands (0.05 - 0.25 mm), 10 - 25%, coarse sands, and 5– 10% gravels. Natural dry density is 1.0 to 1.1 ton/m^3 (63 to 69 lb/cu ft) for material under 4.8 mm grain size and natural moisture content ratio 60%. With samples including 150 mm particles moisture content is 35%. The liquid limit is 40 - 70%, plastic limit 30 - 50%. However, a special liquid limit of 120% was observed. Clayey mineralogical analysis indicated a small hydrated halloysite content which causes the phenomena of "Thixotropy".

The following properties are based on tests of material under 4 mm grain size: dry density 1.2 - 1.5 ton/m^3 (JIS Compaction) (75 - 94 lb/cu ft); direct shear angle of internal friction 10° - 25°; cohesion 0.3 - 1.0 kg/cm^2 (4.3 - 14 lb/sq in); and coefficient of permeability 1×10^{-7} cm/sec. Tests on material to 50 mm indicate dry density of 1.5 - 1.9 ton/m^3 (Modified AASHO) (94 - 118 lb/ft^3) and permeability of 1×10^{-6} to 1×10^{-7} cm/sec.

It is intended that core will contain 55 - 65% of particles 5 - 150 mm in diameter.

Porosity of compacted soils 35 to 40%, compared to 60% in natural condition. Test fills resulted in dry densities of 1.6 - 1.7 ton/m^3 (100 - 106 lb/ft^3) for sheeps foot roller (27.9 kg/cm^2 pressure), and 1.5 - 1.6 ton/m^3 (94 - 100 lb/ft^3) for pneumatic tire roller. Density did not increase with increased number of passes for this material with 6 - 8% higher than optimum moisture content. Permeability for field density was 4.0×10^{-5} to 2×10^{-6} cm/sec.

Selection of Type

A central core design, Fig. 1, was adopted for the following reasons:

a. A central core is considered less susceptible to rupture or cracking due to differential deformation of the embankment, resulting from the steep topography and non-uniform foundation support.
b. An inclined core, if placed at the relatively high moisture content necessary because of prevailing weather conditions, would constitute a zone of relatively low shear strength in the position of a potential slide plane, and thus decrease the stability of the upstream slope.
c. The topography of the site establishes highly desirable locations for the upstream tunnel portals and for the crest of the dam. The space between these two locations is insufficient for a sloping core design.

General Design Considerations

The impervious core has slopes of 10 on 1 to minimize the amount of core material to be placed, since weather conditions suitable for moisture-controlled earth fill operations will be limited to about 100 days per year. During construction, lateral drainage to the outer slopes of the narrow core will prevent the occurrence of excessive pore pressure, even though the material is placed at the upper limit of workable moisture content.

Design Data and Analysis

The design data and the results of the stability analysis are presented in Tables 1 and 2. The adopted design constants are based on laboratory tests made by the Aichi Corporation and on tests of similar materials made by the U. S. Bureau of Reclamation, the U. S. Corps of Engineers, and by Electricite de France. The use of comparative results of similar materials was necessary since Aichi Corporation did not have shear test equipment suitable to use with the materials at Makio—all of which have large percentages of coarse materials. Earthquake acceleration of 0.15 g which will act horizontally, is based on probability studies covering 1300 years, and made by Dr. Kawazumi of Tokyo University. "Earthquake tests on models of rockfill dams" were made for Aichi Corporation by Dr. Niwa of Kyoto University. The method of stability analysis used was that of W. Fellenius in, "Calculation of the Stability of Earth Dams". Stability analysis indicated that talus and terrace deposits on the abutments should be removed.

Foundation Treatment

The pattern of intersecting joints, solution channels and faults crossing the site required extensive and careful treatment. The grout curtain, Fig. 2, will be carried out by stage grouting. Shafts were excavated in the faulted zones six meters deep and twelve meters long. Gunite shall be used as required. The CO_2 gas in the formation will affect ordinary cement, and a special cement is required. The special cement is based on tests and on specifications used by New York City in 1941.

Embankment Construction

Embankment construction is planned to begin in July, 1959. Core material shall contain not larger than 150 mm size particles; its moisture content shall be limited to that which would permit the satisfactory use of construction equipment, and compaction shall be by sheepsfoot roller in 20 cm loose layers. Transition zone shall contain not greater than 600 mm (23 inch) cobbles and not more than 5% by weight passing a No. 200 sieve, and shall be compacted by pneumatic tire rollers. Filter zone shall contain not greater than 200 mm (8 inch) cobbles and shall be compacted by sheeps foot roller at the time of rolling the transition zone. The upstream rockfill shall be made by end dumping or other means with no requirement for compaction or leveling in regular lifts as long as the fill is brought up reasonably level. The downstream rock fill shall be of quarry run sizes of durable rock dumped and bulldozed into place in lifts approximately 2 meters thick, compacted by controlled movement of the hauling and spreading equipment without sluicing.

Investigation, design and construction supervision has been by Erik Floor and Associates, and personnel engaged on this project includes D. J. Bleifuss, P. T. Bennett, J. L. Schnitz, F. A. Nickell and G. C. Gilboy.

ZONE and MATERIAL	① IMPERVIOUS CORE	② SANDGRAVEL FILL	③ ROCK FILL	④ SAND FOUNDATION
DRY DENSITY, T/m³	1.40	2.20	1.65	1.70
MOISTURE CONTENT, %	30	5	0	22
WET WEIGHT, AS PLACED	1.82	2.31	1.65	—
SATURATED WEIGHT	1.88	2.38	2.04	2.08
COHESION	0	0	0	0
ϕ	31°	41.7°	45°	36°
tan ϕ	0.60	0.89	1.00	0.73
EARTHQUAKE ACCELERATION --- 0.15g (Acting Horizontally)				

Note: "T" - Tons metric.

TABLE: 1 MAKIO DAM STABILITY ANALYSIS DESIGN DATA

	P.L.	POOL. EL.	EARTHQUAKE ACCELERATION	S.F.
UP. STREAM SLOPE	1	840 m	0.15 g	1.42
	2	"	"	1.89
	3	860	"	1.48
	4	"	"	1.35
DOWN. STREAM SLOPE	1	880	"	1.45
	2	"	"	1.61
	3	"	"	1.40
	4	"	"	1.52
	5	"	"	1.48

TABLE: 2. MAKIO DAM STABILITY ANALYSIS

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ROCKFILL DAMS: NANTAHALA SLOPING CORE DAM[a]

Discussions by F. L. Lawton and Waldo G. Bowman

F. L. LAWTON,[1] M. ASCE.—Although this paper was originally presented at the October 1948 ASCE Convention, the Society's Power Division is to be congratulated on having made it available for discussion in connection with the Symposium on Rockfill Dams, because the Nantahala Dam is the first, so far as known, to use a thin sloping impervious core.

The Nantahala Dam owes its inception to Col. Growdon's interpretation of the genesis of many glacial lakes, formed by closure of old stream beds by glacial moraines. The conversion of Nature's method of construction to Man's has resulted in the development of a type of dam applicable to difficult sites, having an inherently indefinitely long life, and one which frequently can be constructed at a far cheaper cost than other types.

The settlement data reflects the variation in reservoir level as is the case with many other rockfill dams. It would be interesting if the author could indicate whether there has been any signs of lateral movement of the fill, i.e. from the abutments to the maximum section.

WALDO G. BOWMAN,[2] M. ASCE.—The sloping core rockfill dam discussed in Proc. Paper 1742 (and 1743 and 1744) deserves to be classed as a new type.

Concrete dams can be classified or differentiated one from another according to the structural action involved—gravity, arch, buttress, etc. For rockfills, however, in which the structural action is always of gravity type, some other means of classification must be adopted, and the kind of water barrier used is a logical choice.

Using this means of classification, there are perhaps only four basic types of rockfill dams—rigid central barrier, central earth core, rigid upstream face and flexible upstream face. A further refinement of this classification could be made by taking into account the material of which the barriers are constructed, but the dam type would be essentially the same.

Whether, for example, a rigid upstream face is made of concrete, steel or wood is immaterial so far as dam type is concerned. The same would be true for a flexible face of either asphalt or plastic sheets—if the latter should ever be tried. But the sloping earth core, while basically a flexible upstream facing, introduces a distinctive new principle to water barrier design. This principle changes the character of the dam and, in effect, creates a new type.

The principle involved is, of course, to sandwich a thin sheet of earth between rock filter layers which are graded outward from fine to coarse on both

a. Proc. Paper 1742, August, 1958, by James P. Growdon.
1. Chf. Engr., Power Dept., Aluminium Labs. Ltd., Montreal, Canada.
2. Editor, Engineering News Record, New York, N. Y.

sides of the earth. The downstream filter thus keeps the earth barrier, even though it deforms, from migrating through the dam, while the reverse filter upstream performs a similar function in case of sudden drawdown, while it also protects the earth from wave action. The sloping earth core thus performs its function and maintains its structural integrity by quite different means than do other types of flexible barriers. Beyond this fact it is fashioned from materials ready at hand, which can be a decided advantage.

It seems proper to emphasize the uniquely different character of this flexible upstream barrier, even though its significance will not be missed by many readers of the paper.

There is a second significant aspect of the sloping earth core dams described in this paper. It is that in the short space of a decade a new type dam was proposed, designed, built and followed by the construction of six others. Few new developments in any branch of civil engineering have enjoyed such unusual initial success. And it is particularly notable that the same owner, the Aluminum Company of America, which would not be likely to repeat a design just out of pride in the fact that it originated it, is responsible for six of the dams.

This paper (and 1743 and 1744) gives a remarkably complete record of the sloping earth core dam from inception through design, construction and operating performance. As such, it constitutes a valuable addition to engineering knowledge for which the profession owes the author its gratitude. It might conceivably demonstrate this gratitude by referring to such structures as rockfill dams of the Growdon sloping earth core type.

ROCKFILL DAMS: DAMS WITH SLOPING EARTH CORES[a]

Discussion by Torald Mundal

TORALD MUNDAL,[1] M. ASCE.—The author points out that the design of a rockfill dam with sloping earth core is quite simple. The main rock fill supporting the core and the waterload will provide ample factors of safety if it is dumped at the natural angle of repose. This has been verified by laboratory experiments carried out at the University of California for International Engineering Company. It was concluded from these experiments that a triangular rock fill has a large factor of safety against shearing failure when the fluid pressure is applied to the upstream sloping face of the fill.

Model tests were subsequently carried out by the same laboratory to investigate the earthquake resistance of rockfill dams.[2] It was concluded from these experiments that rockfill dams with clay core are inherently very resistant to earthquakes. It was also indicated that the sloping core type is somewhat more earthquake resistant than the central core type because of its greater rigidity.

The general stability of a sloping core rockfill dam is ordinarily not questioned, but there are details of both design and construction which require very careful consideration and treatment. An appreciable settlement can not be avoided in a dumped rockfill. In order to avoid damage to the core, large abrupt differential settlements must be prevented. The materials going into the main rock fill should, therefore, have uniform settlement characteristics and extensive pockets of inferior materials, particularly concentrations of fine fragments, should not be allowed.

The contact between clay core and abutments requires special care. The abutments should be shaped in such a way that the change in the amount of settlement is gradual. Close control of construction operations, by personnel experienced in this type of work, can not be over emphasized.

The author, who originated large sloping clay core rockfill dams, has incorporated in the Bear Creek Dam the improvements gained in his later extensive experience with this type of dams. This dam is an excellent example of obtaining maximum economy by close coordination of design with construction methods. When these principles are adhered to, a sloping core rockfill dam on a suitable site will compare favorably costwise with other types.

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a. Proc. Paper 1743, August, 1958, by James P. Growdon.

1. Vice-Pres. and Chf. Engr., International Engineering Co., San Francisco, Calif.
2. Earthquake Resistance of Rockfill Dams by Roy W. Clough and David Pirtz, Associate Members ASCE Transactions Paper No. 2939.

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ROCKFILL DAMS: THE PERFORMANCE OF SEVEN SLOPING CORE DAMS[a]

Discussion by F. L. Lawton

F. L. LAWTON,[1] M. ASCE.—The author's use of fuse-plug fills in connection with several developments embodying rockfill dams has no doubt contributed materially to the low development costs achieved. Could the author indicate the criteria used in the design of the fuse plugs? By how much must a fuse plug be over-topped before it will wash out?

The experience at Cedar Cliff and Bear Creek Dams associated with the development of a crack along the crest between the upstream rock blanket and the impervious core is interesting. Would the author care to indicate whether, in his opinion, this is an indication of slippage of the upstream rock blanket on the underlying filter zone or of the filter zone on the impervious core? It is believed these two are the only known instances of such an occurrence although they have been reported for a number of rockfill dams with a central core of earth supported by heavy rock shoulders.

The low capital and annual costs noted for the rockfill dams built under the author's supervision is of considerable importance in relatively inaccessible locations.

a. Proc. Paper 1744, August, 1958, by James P. Growdon.
1. Chf. Engr., Power Dept., Aluminium Labs. Ltd., Montreal, Canada.

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ROCKFILL DAMS: PERFORMANCE OF MUD MOUNTAIN DAM[a]

Discussions by J. B. Cooke and F. L. Lawton

J. B. COOKE,[1] M. ASCE.—The author gives an interesting summary of the high central core rockfill dam. Additional detail on the settlement would be of particular value, since measurements as described by the author, but briefly summarized, are seldom as thoroughly taken.

The profile at Mud Mountain puts the lower half of the dam between steep canyon walls and the abutments are more gradually sloped for the upper half of the dam. A profile of the vertical settlement plotted over this abrupt bedrock profile would be of particular interest. For example, does the crest over the top of the nearly vertical canyon wall settle as a 200-foot dam or as a 400-foot dam, and is there any arching tendency over the narrow canyon that might reduce settlement over that which would occur in a broad 400-foot fill. It has been the writer's observation that the crest settlement profile is much smoother than the foundation profile and, for more gradual profiles than that of Mud Mountain, affected more by the high central fill than the shallower abutment fill. The maximum movement at the highest section seems to drag the adjacent sections of less height.

The cracks between core and shell have been observed on a number of dams and are not very deep. They are generally agreed to be of little concern. An interesting reference on such cracks, aside from this Symposium, is an article by Raul P. Marsal and Enrique Tamez in the Indian Journal of Power and River Development, Vol. VII, No. 5, May 1957, p 18, 5 pp, 2 ff, 2 plates. Cracks which did not occur at Mud Mountain, but which would cause concern in a core rockfill dam would be transverse to the core and in the core near the ends of the crest of the dam. The tendency, particularly of the rockfill, is to develop tension near the abutments and compression near the center. This is most evident in impervious face rockfills and should be less so in sloping and then in central core rockfills as the influence of the rockfill decreases. The tensions may not cause cracks and, if they should, the filters are there to insure healing. However, as rockfill dams become higher, the predictions of movements becomes increasingly important. Few lateral movement measurements on core rockfill dams have been published and those taken on the high Mud Mountain would be valuable.

The downstream movement of crest points would be of interest even though the dam has been filled to only 300 feet of its 400-foot height.

a. Proc. Paper 1745, August, 1958, by Allen S. Cary.

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F. L. LAWTON,[1] M. ASCE.—Despite the fact the settlement in the Mud Mountain Dam was about as anticipated, it would appear rather surprising that shortly after completion of the dam cracks appeared on the crest along the junction between the core and the upstream and downstream transition zones parallel to the axis. It is true the cracks are reported as having been ". . . completely undiscernible at 6 feet depth", but this is not necessarily an indication of no slippage of the shell zones on the transition zones. Would the author care to comment on this point?

It is noted that the rockfill consists of an andesite described as being "tuffaceous to very hard". How has this rockfill withstood weathering?

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ROCKFILL DAMS: WISHON AND COURTRIGHT CONCRETE FACE DAM[a]

Discussion by G. I. Davey

G. I. DAVEY,[1] M. ASCE.—The details of design of the Wishon and Courtright Dams as they compare with the Salt Springs and Lower Bear River Dams reveal that a great deal more attention has been given to the details of construction operations so that savings can be effected by design particularly in relation to side slopes and facing rock. It seems that the thickness of placed rock has been dictated by construction reasons rather than design, in that ample width facilitated dumping methods.

It may be of interest to consider two dams built during 1957 in Australia, one at Mount Isa, Queensland, on the Leichhardt River and the other at Mary Kathleen on the Corella River. The dams are very similar in construction and on similar river systems, about fifty miles apart.

The dams were constructed for water supply purposes, in areas where the rainfall is seasonal and averages about fourteen inches. Evaporation is high and totals about nine feet per annum. Good run-off from the catchment areas is dependent on short period intensive rains which will give a run-off.

Details of the dimensions are set out in Table 1.

It should be noted that the height of these dams was limited by the structure of the river valleys and they could not be substantially higher.

TABLE 1

SITE	CORELLA RIVER	LEICHHARD RIVER
Catchment Area	140 sq. miles	447 sq. miles
Storage	10,600 ac.ft.	61,000 ac. ft.
Estimated maximum flood discharge	30,000 cusecs	60,000 cusecs
Nature of rock foundation	Silicified limestone	Quartzite
Spillway construction	Side cut with automatic gates	Side cut with concrete sill
Height at axis	75'	87'
Length of crest	480'	850'
Side Slopes		
U.S.	1:1¼	1:1¼
D.S.	1:1¼	1:1¼
Facing Placed rock	2'	2'6"
Facing "Gunite"	4" to 3"	5" to 3"

a. Proc. Paper 1746, August, 1958, by Barry Cooke.

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Leichhardt River Dam

The dam is sited on solid rock where the Leichhardt River breaks through a quartzite ridge. The body of quartzite is flanked by silicified shales.

No diversion works were provided. The risk of flooding by out of season rainfall was covered by insurance.

A spillway was excavated on the northern bank from which some 100,000 cubic yards of rock were obtained for dam construction and 100,000 cubic yards dumped to waste. The balance of fill required for the dam was quarried from the quartzite masses adjacent to the dam site. The site was closely drilled and grouted to a depth of 30' and very little grout was injected.

Construction was carried out by dumping from 12 yard trucks loaded by 2-1/2 yd. shovels in the quarries. The fills were limited to a maximum of 30' and generally lesser heights were used. Dumping was controlled to ensure maintenance of the designed slope lines. The fill was constantly trimmed and traversed by a heavy bulldozer. The face of each dumping point was kept wet by constant hosing using 1" nozzle at 100 p.s.i. Each truck delivering rock to the fill paused en route under a 4" diameter nozzle so that the contents of the truck were wetted and the voids in the truck body filled with water. When dumped the wet rocks with water slid down the wetted face of the fill, tended to drag the fill down and maintained velocity in falling to the bottom.

This method of watering was adopted in the first case because of the difficulty of obtaining sufficient water. The Leichhardt River is dry almost the whole year and the only water available was in a small water hole immediately downstream of the dam site. The water pumped from this hole and used in facilitating consolidation was drained back into the water hole and recirculated. The method of filling trucks with water was continued when the excellent consolidation results obtained were noted.

There has been no measurable settlement in the fill since it was completed. Certainly the dam has not yet filled because of low rainfall and run-off though it has some forty feet of water.

The calculated voids in the dam amount to 30% of the total volume.

The dam was faced by selecting stones from the fill and packing in place by hand for a thickness of about 2'. This surface was then covered with a lean (10:1) sand-cement mixture using a cement gun. On this surface formwork and scaffolds were set up and panels of reinforcing mesh placed. The forming held dumb-bell type rubber jointing strips surrounding each panel, which was approximately 30' square. The surface sand-cement mixture (3:1) was then placed by cement gun, the thickness varying from 6" to 4". After placing the surface was sprayed with curing wax covered with sheets of hessian and kept damp for three weeks.

Some fin cracks have appeared in the surfacing where exposed to the air but they are unlikely to allow of leakage.

The dam has performed very satisfactorily to date and there is no evidence of leakage.

Corella River Dam

The site is in solid rock, and was drilled and grouted. Very little grout could be injected to a depth of 30'. (It has been found by subsequent drilling that isolated cavities occurred in the silicified limestone and additional grouting has been carried out).

Again, no diversion works were constructed as the river bed is dry except in times of rain, and it was proposed to construct the dam in the period between June and November when rain rarely falls.

Again, a spillway was excavated, on the southern bank and the whole of the material required for the construction of the dam was obtained from the cut. The total excavation was 100,000 cubic yards and the fill in the dam was 110,000 cubic yards.

The fill was made by dumping from 12 yd. trucks loaded by a 3 C. yd. shovel. The maximum depth of fill at any one time was 30' but generally 20' was used. The dumping was done with care to maintain the designed slopes, and the procedure was exactly as described for the Leichhardt River dam except that the rock was sluiced as it was dumped from the truck and on the dump using two 1-1/2" nozzles. Generally the quantity of water was equal to twice the rock volume. Water for sluicing was obtained from a large water hole immediately upstream of the dam site. The trucks were not filled with water, as at Leichhardt River Dam, until the fill was almost finished, when the idea of filling the truck was developed.

The voids in the fill were calculated at 32% of the total volume.

The facing of the Corella River Dam was the same as that at the Leichhardt River and the methods used were identical. Rubber water stops were not used in the joints, a bitumenous filler was used in formed grooves, the reinforcement being continuous through the joint.

The dam has performed well though it has not yet completely filled. There have been minor cracks in the surfacing which has been sealed with liquid grout.

No settlement has been measured in the fill since it was completed in December 1956, and the maximum settlement recorded during construction on any stage of the fill was 0.02 feet.

Leakage has been small but consistent and has been proven to pass through the abutments where further grouting has been carried out which has reduced the leakage to 1.5 cusecs.

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ROCKFILL DAMS: THE PARADELA CONCRETE FACE DAM[a]

Discussion by J. Barry Cooke

J. BARRY COOKE,[1] M. ASCE.—The authors have presented a valuable and comprehensive paper on what is presently the world's highest concrete face rockfill dam. They have graciously made references to the influence of the design and performance of Pacific Gas and Electric Company's Salt Springs and Lower Bear River Dams (Proc. Paper 1737) on the design of their Paradela Dam and on the selection of the type of dam. However, the design and construction of P.G. and E.'s Wishon and Courtright Dams proceeded concurrently with that of Paradela Dam, and it is the purpose of this discussion to compare briefly the design and construction of P.G. and E.'s recent concrete face rockfill dams with that of Paradela Dam. The design and construction of the Wishon and Courtright Dams is presented in Paper 1746 of this Symposium.

Foundation and Cutoff

The weathered granite foundation of Paradela is very inferior to the excellent granite bedrock at the sites of the P.G. and E. dams. The use of a concrete bed under the placed rock and the use of the gallery and drains in the cut off wall at Paradela is considered a prudent and logical difference. At Lower Bear River Dam the placed rock was bedded in concrete at some local zones of adverse slope or inferior quality of bedrock to improve effectiveness of grouting, provide insurance against a blowout, and to increase sliding resistance.

Shape of Dam

The writer agrees with the authors' reasons for the moderate curvature in plan and that curvature is important in the central and lower portion of the dam. However, the term "arch action" may not best describe the favorable action of the curvature. The curvature should be such that the resultant settlement normal to the face will always leave a slight upward curvature. More than this minimum curvature in the lower and central area of the face is undesirable since the high water pressures, acting normal to the face, tend to push the face slabs toward the center of the dam. Greater than minimum curvature at and near the crest is desirable for appearance and is satisfactory since water pressures are nominal in the upper portion of the dam. A line

a. Proc. Paper 1747, August, 1958, by Luis Henrique Fernandes, Edgard de Oliveira, and Nuno de Vasconcelos Porto.

1. Supervising Civ. Engr., Pacific Gas and Electric Co., San Francisco, Calif.

across the face will shorten one to several inches due to the "flattening" of the face caused by the settlement. The placed rock lateral contacts are certainly not adequate to even tend to resist such shortening of one to several inches in a several hundred to a thousand foot length by "arch action". What is visualized to occur is a closing of joints due to the distance between adjacent vertical joint lines being shorter after settlement. The amount of each joint closing from this cause may be computed from the differential movement of each slab between joint lines. At the center of the face where both ends of the slab move about the same distance there is no joint closing from this cause. Near the abutment where differential movement is greatest the joint closure due to this cause is greatest. The tendency to close particularly the vertical joints near the abutments favorably tends to offset the opening of those joints by the basic rockfill movements described by the authors later in their paper.

The upstream face slope at Wishon, 1.3:1 to 1:1, is steeper than the 1.3:1 slope of Paradela and is economically constructed with the use of "face lifts". Substantial cost savings were made at Wishon and Courtright due to the steep upstream slope and to the use of face lifts. The moderately more conservative upstream face slope at Paradela may in part be due to the authors' not wanting to break other precedents along with the very major precedent they were establishing in maximum height of this type of dam. The downstream slopes of all three dams have resulted essentially in 1.3:1 slopes in the upper portion and 1.4:1 slopes in the lower portion even though layouts and construction lifts were different. This occurs because of the characteristic of rockfills dumped from high lifts to be steeper at the top than near the bottom of the dumped slope, as described by the authors.

The Placed Rock

The placed rock at Paradela is substantially thicker than that at Wishon and Courtright. Comparative thicknesses for Courtright and Paradela are respectively: at top, 7.8 and 10 feet; at 100 feet down, 7.8 and 15 feet and at 290 feet down 11.6 and 24.5 feet. The writer believes that the modern well sluiced and interlocked rockfill of large rocks is as competent as placed rock for upstream slopes of 1:1 and flatter, and that a minimum thickness of placed rock is satisfactory. The major saving at Wishon over previous designs was in the thin placed rock, since placed rock of Wishon Specifications and in the United States costs about 5 times as much as dumped rock. In Portugal the ratio may be much smaller. It would be of interest to know the production rate in cubic yards per 8-hour crane shift at Paradela as well as the above cost ratio. Placed rock also tends to govern the construction schedule unless it is of minimum thickness, and thus adds costs that are not obviously chargeable to the placed rock. The writer wonders if the much thicker placed rock at Paradela might not be due to lower cost in Portugal and the unprecedented height of dam and not to an emphasis on its importance by the designers.

The authors describe the method of placing rock from cranes on the placed rock and the difficulties in supplying the cranes with rock and in removing the fines that accumulate on the surface of the dumped rock as a result of supplying the placed rock from the face of the dumpted rock. Experience at Salt Springs and Lower Bear River was similar; the "face lift" method was adopted at Wishon and Courtright to avoid those troubles, give greater mobility to the cranes and in general to lower the cost of placing rock.

Rockfill

The authors mention "extensive slides of rockfill . . . two minutes . . . etc." The writer has observed surface slides during dumping where the whole dump surface may be in motion in a re-arrangement of surface rock. The dumped slope is very irregular and contains zones of steeper than the average dumped slope. Occasionally a load or a large sliding rock would set off a surface slide. However, from the writer's observations these slides are on the surface and they do not endanger the rear tires of the dump truck. They certainly provide a valuable service in compaction of the fill and they testify to the high shear capability of the rockfill since the energy and vibration do not start a more substantial than surface slide. Does this agree with the nature of slides on the particularly high lifts of Paradela?

Impervious Membrane

The thickness of slab and reinforcing is much the same for all high dams of this type and experience has indicated the accepted design to give satisfactory service and long life. The design features that can vary greatly are joint arrangement, spacing and design. The factors of height of dam and of reservoir operation are of major importance in their influence on the economic spacing and design of joints. The magnitude of movements increases rapidly in the 150 to 350 foot height range. More care in design and higher cost is necessary for joints that may be unwatered only by impairing normal reservoir operation or for joints that may never be unwatered. Wishon reservoir will normally be drawn down to one-half depth each year, but could be completely unwatered in October of any year should it be necessary. Courtright reservoir level will normally be lowered to only 30 feet below full reservoir level each year with dry cycles requiring nearly full unwatering once in perhaps six years. In comparing the joint spacing and details of Paradela and Wishon, the joints at Paradela are more closely spaced, more elaborate and better. The application of the above basic design criteria as well as the lower labor costs at Paradela are believed to be the reasons.

An important consideration regarding the impervious face of this type of dam is that cracks and leakage do not affect safety and, should they occur, the membrane is accessible. If leakage occurs, and represents an economic loss, the trouble can be located and can be repaired under water, or in the dry during a normal or specially scheduled lowering of the reservoir. Since the main settlement and movement of the dam takes place during and shortly after the first filling, repairs made after that time should be permanent, and later leakage is unlikely.

Face Slab Thickness and Reinforcing

The concrete face at Paradela has the same reinforcing as used at Wishon and other high dams of the concrete face type. The theoretical thickness of slab at Paradela of $1 + 0.00735h$ compares with $1 + 0.0063h$ for Wishon. However, the difference in manner of placing rock results in a greater actual average thickness at Wishon. At Paradela the placed rocks are laid with surfaces approximately flush to the plane of the face which reduces the amount of concrete outside the design thickness. The faces at Dix River and Salt Springs were similarly well laid when labor cost in this country was relatively low, and concrete relatively expensive. The economic practice at Wishon was

to use large rocks and high capacity cranes and to use minimum hand labor. This results in a stepped and irregular surface and high production per crane shift, that gives minimum cost of placed rock with some penalty in excess face concrete.

Joint Spacing

The vertical joint spacing of 49.3 feet at Paradela is closer than the 60 feet spacing at Wishon. The horizontal joint spacing is 32.9 to 49.3 feet at Paradela and 32.8 to 62.4 feet at Courtright. The top two slabs at Wishon are 77.8 feet long since the broad even profile should not produce differential settlements in the upper portion of the dam and the economy is considered possible. The maximum spacing of joints at Wishon and Courtright is considered important economically to make large pours possible and to minimize the cost of joints.

Joint Ribs

Joint ribs at Wishon were a minimum, the requirement being only a smooth asphalted surface of specified width. This effected economies in simplifying the placing of rock and reducing concrete quantities.

Joint Intersections

The prefabricated "seals crossing piece", Fig. 4 of author's paper, is similar to that successfully used at Lower Bear River Dam No. 1. If the intersecting joints both open, such a crossing is essential to prevent tearing copper. If only one joint opens, or if both joints close, such a crossing is not necessary. At Paradela the 1.3:1 face slope and the dumping of the rock of Stage 3 over Stage 2 tend to produce opening of the upper horizontal joints and measurements of Fig. 8 of author's paper indicates horizontal joint openings to have occurred. Also, at Paradela the hinge slabs having a "soft joint" on all sides can move to permit opening of both intersecting joints. The "seals crossing piece" is necessary for many of the joint intersections at Paradela. At Wishon the dumping of the rock and the 1:1 slope at the upper portion of the face was predicted to result in closing of all horizontal joints. The "perimetral" and "connecting" joints at Wishon are cold joints and the U-shaped copper of the perimetral joints is carried through the intersection to permit perimetral joints to open if they choose to. The "seals crossing piece" was considered not to be necessary at Wishon and Courtright and was not used.

Joints

The writer considers the Paradela joints to be particularly well designed. The location of the copper above slab centerline in all joints at Paradela is better than below. Any loading that might develop would be on the bottom edge of the slab and not on the top edge. To have the main reinforcing below the copper and in the thicker portion of the slab that is divided by the copper is fundamentally desirable and costs no more. The important vertical joints are essentially of the same design. The horizontal joints at both Paradela and Wishon are soft joints, which proved very successful at Lower Bear River No. 1, but the Wishon joints are more simple, having a small Z shaped copper, since it was predicted that the Wishon horizontal joints will all close. The use of compressible material under the slab at the cut off joint is considered

an improvement and was used at both Paradela and Courtright. A difference in design thinking is that the Paradela perimetral and connecting joints are "soft" joints, whereas at Wishon they are cold joints to provide lines of zero shear and moment and to resist movement, thus minimizing movements of the main slabs. However, the joints are asphalt mopped and do contain U shaped copper with some asphalt filler to permit opening and offsetting. The cold cut off and perimetral joint design is necessary with the simple horizontal joint and the lack of a "seals crossing piece" at intersections, but may have a possibility of crushing. The soft perimetral and connecting joints at Paradela may result in greater joint movements but all the joints are so well designed and constructed that they can in all probability take the movement without trouble.

An engineer reviewing and comparing the Papers of this Symposium on the concurrently designed Paradela and Wishon-Courtright Dams, might conclude that there are substantial differences in the design thinking behind the two dams. It is the writer's belief that the differences are not at all fundamental but are logical products of the differences in foundation conditions, heights, labor costs, reservoir operating schedules and other factors. The unprecedented height of Paradela would certainly have a major influence on design and tend to prevent the departures from previous practice made at Wishon and Courtright. It is hoped that the author's closing discussion will clearly point out any differences in fundamental design consideration, should there be any. Also, it would be of value if the authors could provide crest settlement measurements and other observations taken after the reservoir was filled and unwatered, which occurred since the writing of their paper.

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ROCKFILL DAMS: THE PARADELA DAM—FOUNDATION TREATMENT[a]

Discussion by F. L. Lawton

F. L. LAWTON,[1] M. ASCE.—The author has clearly recognized the two peculiar features associated with the sealing of a permeable foundation below the cut-off wall in a deck-type rockfill dam with concrete face. The grouting scheme adopted for Paradela Dam would appear to adequately deal with these two features, i.e.

1. The full hydraulic head from the reservoir at capacity must be met by a very small zone of soil or rock around the contact surface of the cut-off wall.
2. It is difficult to detect and to stop grout leakage at the base of the rockfill.

It would appear that one very effective means of stopping grout leakage at the base of the rockfill would be the placement of grout covering the desired area with a grout blanket of suitable thickness. This has been done in some cases although perhaps not with deck-type rockfill dams using a concrete facing.

The concrete cut-off with gallery used at Paradela Dam would appear to be a rather expensive method of achieving a satisfactory grout curtain. Would the author care to comment on the relative cost of the grout curtain carried out from the gallery in the concrete cut-off as compared with one carried out in the open?

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a. Proc. Paper 1748, August, 1958, by Walter J. Weyermann.

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PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY 7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

VOLUME 84 (1958)

APRIL: 1580(EM2), 1581(EM2), 1582(HY2), 1583(HY2), 1584(HY2), 1585(HY2), 1586(HY2), 1587(HY2), 1588 (HY2), 1589(IR2), 1590(IR2), 1591(IR2), 1592(SA2), 1593(SU1), 1594(SU1), 1595(SU1), 1596(EM2), 1597(PO2), 1598(PO2), 1599(PO2), 1600(PO2), 1601(PO2), 1602(PO2), 1603(HY2), 1604(EM2), 1605(SU1)[c], 1606(SA2), 1607(SA2), 1608(SA2), 1609(SA2), 1610(SA2), 1611(SA2), 1612(SA2), 1613(SA2), 1614(SA2)[c], 1615(IR2)[c], 1616 (HY2)[c], 1617(SU1), 1618(PO2)[c], 1619(EM2)[c], 1620(CP1).

MAY: 1621(HW2), 1622(HW2), 1623(HW2), 1624(HW2), 1625(HW2), 1626(HW2), 1627(HW2), 1628(HW2), 1629 (ST3), 1630(ST3), 1631(ST3), 1632(ST3), 1633(ST3), 1634(ST3), 1635(ST3), 1636(ST3), 1637(ST3), 1638(ST3), 1639(WW3), 1640(WW3), 1641(WW3), 1642(WW3), 1643(WW3), 1644(WW3), 1645(SM2), 1646(SM2), 1647 (SM2), 1648(SM2), 1649(SM2), 1650(SM2), 1651(HW2), 1652(HW2)[c], 1653(WW3)[c], 1654(SM2), 1655(SM2), 1656(ST3)[c], 1657(SM2)[c].

JUNE: 1658(AT1), 1659(AT1), 1660(HY3), 1661(HY3), 1662(HY3), 1663(HY3), 1664(HY3), 1665(SA3), 1666 (PL2), 1667(PL2), 1668(PL2), 1669(AT1), 1670(PO3), 1671(PO3), 1672(PO3), 1673(PL2), 1674(PL2), 1675 (PO3), 1676(PO3), 1677(SA3), 1678(SA3), 1679(SA3), 1680(SA3), 1681(SA3), 1682(SA3), 1683(PO3), 1684 (HY3), 1685(SA3), 1686(SA3), 1687(PO3), 1688(SA3)[c], 1689(PO3)[c], 1690(HY3)[c], 1691(PL2)[c].

JULY: 1692(EM3), 1693(EM3), 1694(ST4), 1695(ST4), 1696(ST4), 1697(SU2), 1698(SU2), 1699(SU2), 1700(SU2), 1701(SA4), 1702(SA4), 1703(SA4), 1704(SA4), 1705(SA4), 1706(EM3), 1707(ST4), 1708(ST4), 1709(ST4), 1710 (ST4), 1711(ST4), 1712(ST4), 1713(SU2), 1714(SA4), 1715(SA4), 1716(SU2), 1717(SA4), 1718(EM3), 1719 (EM3), 1720(SU2), 1721(ST4)[c], 1722(ST4), 1723(ST4), 1724(EM3)[c].

AUGUST: 1725(HY4), 1726(HY4), 1727(SM3), 1728(SM3), 1729(SM3), 1730(SM3), 1731(SM3), 1732(SM3), 1733 (PO4), 1734(PO4), 1735(PO4), 1736(PO4), 1737(PO4), 1738(PO4), 1739(PO4), 1740(PO4), 1741(PO4), 1742 (PO4), 1743(PO4), 1744(PO4), 1745(PO4), 1746(PO4), 1747(PO4), 1748(PO4), 1749(PO4).

SEPTEMBER: 1750(IR3), 1751(IR3), 1752(IR3), 1753(IR3), 1754(IR3), 1755(ST5), 1756(ST5), 1757(ST5), 1758(ST5), 1759(ST5), 1760(ST5), 1761(ST5), 1762(ST5), 1763(ST5), 1764(ST5), 1765(WW4), 1766(WW4), 1767(WW4), 1768(WW4), 1769(WW4), 1770(WW4), 1771(WW4), 1772(WW4), 1773(WW4), 1774(IR3), 1775 (IR3), 1776(SA5), 1777(SA5), 1778(SA5), 1779(SA5), 1780(SA5), 1781(WW4), 1782(SA5), 1783(SA5), 1784 (IR3)[c], 1785(WW4)[c], 1786(SA5)[c], 1787(ST5)[c], 1788(IR3), 1789(WW4).

OCTOBER: 1790(EM4), 1791(EM4), 1792(EM4), 1793(EM4), 1794(EM4), 1795(HW3), 1796(HW3), 1797(HW3), 1798(HW3), 1799(HW3), 1800(HW3), 1801(HW3), 1802(HW3), 1803(HW3), 1804(HW3), 1805(HW3), 1806 (HY5), 1807(HY5), 1808(HY5), 1809(HY5), 1810(HY5), 1811(HY5), 1812(SM4), 1813(SM4), 1814(ST6), 1815 (ST6), 1816(ST6), 1817(ST6), 1818(ST6), 1819(ST6), 1820(ST6), 1821(ST6), 1822(EM4), 1823(PO5), 1824 (SM4), 1825(SM4), 1826(SM4), 1827(ST6)[c], 1828(SM4)[c], 1829(HW3)[c], 1830(PO5)[c], 1831(EM4)[c], 1832(HY5)[c]

NOVEMBER: 1833(HY6), 1834(HY6), 1835(SA6), 1836(ST7), 1837(ST7), 1838(ST7), 1839(ST7), 1840(ST7), 1841(ST7), 1842(SU3), 1843(SU3), 1844(SU3), 1845(SU3), 1846(SU3), 1847(SA6), 1848(SA6), 1849(SA6), 1850(SA6), 1851(SA6), 1852(SA6), 1853(SA6), 1854(ST7), 1855(SA6)[c], 1856(HY6)[c], 1857(ST7)[c], 1858 (SU3)[c].

DECEMBER: 1859(HY7), 1860(IR4), 1861(IR4), 1862(IR4), 1863(SM5), 1864(SM5), 1865(ST8), 1866(ST8), 1867 (ST8), 1868(PP1), 1869(PP1), 1870(PP1), 1871(PP1), 1872(PP1), 1873(WW5), 1874(WW5), 1875(WW5), 1876 (WW5), 1877(CP2), 1878(ST8), 1879(ST8), 1880(HY7)[c], 1881(SM5)[c], 1882(ST8)[c], 1883(PP1)[c], 1884(WW5)[c], 1885(CP2)[c], 1886(PO6), 1887(PO6), 1888(PO6), 1889(PO6), 1890(HY7), 1891(PP1).

VOLUME 85 (1959)

JANUARY: 1892(AT1), 1893(AT1), 1894(EM1), 1895(EM1), 1896(EM1), 1897(EM1), 1898(EM1), 1899(HW1), 1900(HW1), 1901(HY1), 1902(HY1), 1903(HY1), 1904(HY1), 1905(PL1), 1906(PL1), 1907(PL1), 1908(PL1), 1909(ST1), 1910(ST1), 1911(ST1), 1912(ST1), 1913(ST1), 1914(ST1), 1915(ST1), 1916(AT1)[c], 1917(EM1)[c], 1918(HW1)[c], 1919(HY1)[c], 1920(PL1)[c], 1921(SA1)[c], 1922(ST1)[c], 1923(EM1), 1924(HW1), 1925(HW1), 1926 (PL1), 1927(HW1), 1928(HW1), 1929(SA1), 1930(SA1), 1931(SA1), 1932(SA1).

FEBRUARY: 1933(HY2), 1934(HY2), 1935(HY2), 1936(SM1), 1937(SM1), 1938(ST2), 1939(ST2), 1940(ST2), 1941(ST2), 1942(ST2), 1943(ST2), 1944(ST2), 1945(HY2), 1946(PO1), 1947(PO1), 1948(PO1), 1949(PO1), 1950(HY2)[c], 1951(SM1)[c], 1952(ST2)[c], 1953(PO1)[c], 1954(CO1), 1955(CO1), 1956(CO1), 1957(CO1), 1958 (CO1), 1959(CO1).

MARCH: 1960(HY3), 1961(HY3), 1962(HY3), 1963(IR1), 1964(IR1), 1965(IR1), 1966(IR1), 1967(SA2), 1968 (SA2), 1969(ST3), 1970(ST3), 1971(ST3), 1972(ST3), 1973(ST3), 1974(ST3), 1975(ST3), 1976(WW1), 1977 (WW1), 1978(WW1), 1979(WW1), 1980(WW1), 1981(WW1), 1982(WW1), 1983(WW1), 1984(SA2), 1985(SA2)[c], 1986(IR1)[c], 1987(WW1)[c], 1988(ST3)[c], 1989(HY3)[c].

APRIL: 1990(EM2), 1991(EM2), 1992(EM2), 1993(HW2), 1994(HY4), 1995(HY4), 1996(HY4), 1997(HY4), 1998 (SM2), 1999(SM2), 2000(SM2), 2001(SM2), 2002(ST4), 2003(ST4), 2004(ST4), 2005(ST4), 2006(PO2), 2007 (HW2)[c], 2008(EM2)[c], 2009(ST4)[c], 2010(SM2)[c], 2011(SM2)[c], 2012(HY4)[c], 2013(PO2)[c].

c. Discussion of several papers, grouped by divisions.

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